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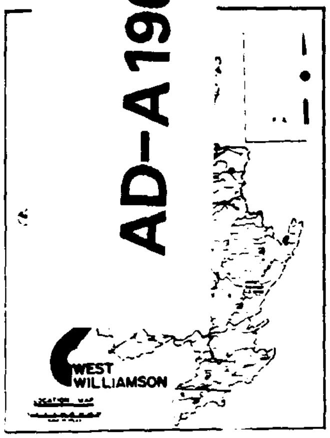

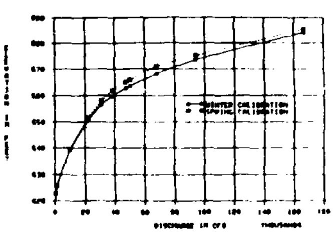

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LEVEE DESIGN PROFILES FOR THE WILLIAMSON, WEST VIRGINIA, FLOOD PROTECTION PROJECT

by

David T. Williams

Hydraulics Laboratory

DEPARTMENT OF THE ARMY
Waterways Experiment Station, Corps of Engineers
PO Box 631, Vicksburg, Mississippi 39180-0631

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This report describes a hydraulic study of the Tug Fork River, West Virginia, from the Kermit gage to the town of Williamson. Williamson is the site of a major flood protection project and is to be protected to the Standard Project Flood (SPF). The objectives of the study were to aid in updating and recalibrating the original HEC-2 model of the US Army Engineer District, Huntington, identify and incorporate into the model important hydraulic parameters, develop a procedure for extrapolation of the stage-discharge rating curve to the SPF discharge at Williamson, and determine the SPF water-surface profile under project conditions. Keywords: hydrology; Flood control;

With mylar overlays of the maps as guides, overbank reach lengths of the HEC-2 model were adjusted to reflect the length of the centroids of the flow lines. Representation of buildings was made by adjustments to the geometry points, and the most recent bridge

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surveys were used to update the geometry. Geometric adjustments were also made in the town of Williamson to allow flow through the central business district (CBD).

Because the 1977 discharge estimates (flood of record) of the Huntington District and the US Geological Survey (USGS) differed (117,000 cfs versus 94,000 cfs, respectively), different calibrations of the HEC-2 model were performed based upon the two discharges. Examination of the calibrated 1984 flood water-surface elevations resulted in reexamination of the rainfall data, and the 1984 flood was reconstructed. This changed the main stem discharge from 82,000 cfs to 58,000 cfs for the 1984 flood.

Analyses of the detailed USGS discharge/velocity measurements of the 1984 flood indicated significant flow through the Williamson CBD during the 1977 flood. The HEC-2 model was adjusted to reflect the geometry of the buildings and streets. Checks were made to assure that the side flow over the existing floodwall into the CBD was sufficient to meet the CBD conveyance potential.

Analyses of the 1984 and 1977 floods and pre-1984 USGS rating curves suggested that winter and spring foliage conditions produced different rating curves which resulted in the adoption of winter and spring rating curves. The rating curves were extrapolated to the SPF by the following method:

a. Plot composite channel roughness k_s versus water-surface elevation for known conditions.

b. Extrapolate k_s to the estimated water-surface elevation for the SPF event but not less than the minimum k_s for the bed material grain size under plane bed conditions.

c. Estimate the water-surface elevation of the SPF event and from the extrapolated curve, determine the k_s .

d. For the k_s and estimated water-surface elevation, calculate the Manning's n value and input to the HEC-2 model.

e. Check the resulting HEC-2 water-surface elevation against the estimated elevation and, if different, make additional estimates until the computed and estimated water-surface elevations match.

The HEC-2 model was modified for project conditions by adjusting the geometry and roughness associated with floodwalls and channel modifications and taking into consideration the containment of the water discharge within the floodwalls and levees.

Preface

The levee design profile study of the Williamson, West Virginia, flood protection project documented by this report was performed for the US Army Engineer District, Huntington.

The study was conducted in the Hydraulics Laboratory of the US Army Engineer Waterways Experiment Station (WES) during the period March 1985 to September 1985 under the direction of Messrs. H. B. Simmons and F. A. Herrmann, former and present Chiefs of the Hydraulics Laboratory, and M. B. Boyd, Chief of the Hydraulic Analysis Division. The project engineer for this study was Mr. David T. Williams, Math Modeling Group, who also wrote the report. Major efforts in the application and postprocessing of the HEC-2 modeling were provided by Messrs. Ken Halstead and Coy Miller of the US Army Engineer District, Huntington. Mr. Glenn Drummond of the US Army Engineer Division, Ohio River, periodically reviewed the progress of the study and discussed interim results.

COL Dwayne G. Lee, CE, is the Commander and Director of WES. Dr. Robert W. Whalin is the Technical Director.

Contents

Conversion Factors, Non-SI to SI (Metric)
Units of Measurement

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
cubic feet	0.02831685	cubic metres
feet	0.3048	metres
miles (US statute)	1.609347	kilometres
square miles (US statute)	2.589998	square kilometres

LEVEE DESIGN PROFILES FOR THE WILLIAMSON, WEST VIRGINIA, FLOOD PROTECTION PROJECT

Introduction

Study objectives

1. At the request of the Huntington District (ORH), the Hydraulic Analysis Division (HAD) of the Waterways Experiment Station (WES) aided in a comprehensive reanalysis of water-surface profiles for the Williamson, West Virginia, flood protection project. Specific tasks were as follows:

   a. Evaluate the existing HEC-2 numerical model for water-surface profiles and recalibrate as necessary;

   b. Identify the hydraulic parameters which affect the water-surface profiles and incorporate into the model;

   c. Develop a procedure for extrapolating the stage-discharge rating curve at Williamson to the Standard Project Flood (SPF) for existing conditions;

   d. Determine the water-surface profile for the SPF for the modified conditions.

This report documents the role of WES in the study.

Basin description

2. The Tug Fork River, Figure 1, originates in the southeastern part of West Virginia, and in the southwestern part of Virginia among the mountains forming the divide between the Tug Fork River Basin on the north and the Clinch River Basin on the south. The Tug Fork River Basin is bounded on the east by the Guyandot River and Twelvepole Creek Basins and on the west by the Levisa Fork River Basin.

3. The total area of the Tug Fork River Basin is 1,559 square miles, which accounts for 36 percent of the Big Sandy River Basin. Sixty percent of the Tug Fork River Basin is in West Virginia, 31 percent in Kentucky, and 9 percent in Virginia. The Tug Fork River is about 155 miles long and flows in a northwesterly direction. It joins Levisa Fork River, forming the Big Sandy River at Louisa, Kentucky. The Big Sandy flows for 26.8 miles to enter the Ohio River at Callettsburg, Kentucky.

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* A table of factors for converting non-SI units of measurement to SI (metric) units is found on page iv.

4. The Tug Fork River Basin lies wholly within the physiographic province known as the Appalachian Plateau. Although the topography and drainage lines of portions of this province have been modified by continental ice sheets, the Tug Fork watershed is generally rugged and the area is well dissected. Over most of the area, the main streams and their many tributaries flow in deep, narrow, sinuous valleys between steep-side ridges. In the headwater regions the terrain is mountainous, whereas in the lower portion of the area the valleys are relatively wide and the hills gentle and rounded. Williamson is located in the lower third of the basin where the valley is 800 to 900 ft wide.

5. The channel is alluvial with a bottom width from 125 to 200 ft. Banks are stable with heights ranging up to 25 ft above low water. Bed sediments are sands and gravels. Vegetation, predominantly conifers, lines both banks and floodplains except where cleared for agricultural and industrial purposes.

Field reconnaissance

6. A field trip to Tug Fork was conducted on 27 March 1985. The trip began in Huntington, W. Va., at 8:00 a.m. under relatively clear and sunny conditions which prevailed throughout the day. The first stop was at 9:20 a.m. at the highway bridge in Kermit, W. Va., and the trip followed the river to a point about 5 miles upstream of Williamson, a total length of 30 miles. The channel, floodplains, highways, and bridge crossings were points of interest. Hydraulic roughness, constrictions, bed sediments, and bedrock were items of interest.

7. The streambed width at Kermit is 125 to 150 ft with very little constriction at the bridge. Both banks were generally covered with brush and trees from the water line to the top of bank. The trees were 50 to 100 ft high with the lower branches about 20 ft from the base of the tree. The trees and brush were completely devoid of leaves. The trees were fairly mature and stable with only a few fallen into the channel. The bank material near the streambed was mostly fine to medium sand with no appreciable clay and very little silt. The bank slopes were generally 1V on 1H to 1V on 3H. Some rock outcrops were observed but they were at intermittent points and not prevalent throughout this reach of the Tug Fork River. Land use on the overbanks included wooded areas, grassy pastures, completely cleared areas with row crops, and urbanized areas. Very little sand deposition was observed on the

banks except at a location a few miles upstream from Williamson. At that point, sand had deposited on the outside of the bend, and immediately downstream a gravel/sand bar was observed in the middle of the channel.

HEC-2 Numerical Model Development

Original model

8. The original HEC-2 numerical model was developed by ORH. It extended from the USGS gage near Kermit, W. Va., to the central business district (CBD) of Williamson, W. Va. The bulk of the original geometric input data was obtained from orthophoto mapping. The aerial photography was flown in April 1975 and mapped at 5-ft contour intervals on a scale of 200 ft/in. Spot elevations were added in flat areas and at points of interest. ORH hydraulic design personnel determined appropriate locations for the valley cross sections and extracted the overbank data from the maps. Field surveys were conducted to obtain bridge and hydrographic data which were used, along with the overbank data, to form the HEC-2 numerical model.

9. Stream discharge data, high-water marks, and the rating curve at Kermit and Williamson were used to develop the water-surface profiles as described and illustrated in Tug Fork Valley, West Williamson Floodwall FDM No. 3 (1984) and Tug Fork Valley, Williamson Floodwall, FDM No. 5 (1984). Left and right limits of the channel were placed, generally, midway between the channel bottom and top of the streambank. The HEC-2 model assumed the default coefficients for expansion and contraction, 0.3 and 0.1, respectively. Channel and overbank Manning's "n" were calibrated to reproduce high-water marks based upon the 1977 flood discharge of 94,000 cfs. The resulting n-values were 0.040 in the channel and 0.060 to 0.067 in the overbanks. These values were used for the entire length of the model. The n-values were varied for other discharges by using the multiplier option in HEC-2. Both the channel and overbank n-values are multiplied by the same factor. These n-values are shown in Table 1.

Adjustments to the original model

10. Cross-section locations were traced on mylar along with the left and right channel limits and the estimated limits of flooding during the SPF. Flow lines for the overbanks were estimated and sketched between consecutive cross sections. Cross-section plots were studied to estimate the

location of the centroid of flow area for each overbank for the SPF profile. These centroids were plotted on the overlays and connected by lines paralleling the flow lines. While most of the original reach lengths were verified in this exercise, some required adjustment. Generally, the changes consisted of increasing the reach length on the outside of a bend and decreasing the length on the inside.

11. During the development of the mylar overlays, each cross section was checked for reasonable representation of the reach it represents. For example, a building was removed from a cross section if it were the only building on that overbank within that reach. Building representations were left in the cross sections in cases where several buildings were found in a row.

12. Minor revisions to delete small "dead" storage areas were made as appropriate. The channel portion of the cross section at the stream-gaging station in Williamson was modified to reflect the geometric data recorded by the USGS during their discharge measurement, number 157, of the May 1984 flood event. This geometry was accepted as a reasonable "average" shape to which the channel might conform during the SPF event.

13. From the analysis of overbank flow at Williamson during the 1977 flood event (described later), modifications were made to lower the elevations representing the streets in the CBD from top of the floodwall to the actual street elevations. The original HEC-2 numerical model eliminated flow area in the streets below the top of the existing floodwall. The change in street elevations was 3 to 5 ft lower.

14. The bridge sections in the original HEC-2 numerical model had trapezoidal approximations of the waterway openings. Survey notes were compared with those sections and most appeared to be reasonable representations. However, since the actual survey data were available, it was incorporated into the numerical model. Also included in this change was the model representation of the bridge piers and the low chord and roadway elevations of the bridge. Adjustments were then made to the "special bridge" input data for approximating the trapezoidal opening for pressure flow as well as the weir flow/pressure flow threshold elevations. The orifice, weir, and pier loss coefficients were checked and found to conform with recommendations in the HEC-2 User's Manual.

Estimation of the 1977 Flood Discharge

15. During the 1977 flood, all the gages along Tug Fork were destroyed and direct discharge measurements were not obtained. The USGS, using a Manning's "n" of 0.045, applied the slope-area method and calculated a peak discharge of 94,000 cfs at Williamson. ORH estimatd a discharge of 117,000 cfs for this event by applying rainfall to previously developed unit hydrographs, combining the runoff from subwatersheds and routing to Williamson. Since this event is the flood of record, the selection of peak discharge for model calibration makes a significant difference in the subsequent extrapolation to SPF conditions. Because of the difference, two calibrations were performed for the 1977 flood: one based on 94,000 cfs and the other on 117,000 cfs. These will be referred to as the 94,000- and 117,000-cfs calibrations, subsequently.

Identification of Significant Hydraulic Parameters and Conditions

Reconstitutions of 1977 and 1984 flood profiles with HEC-2

16. Using the table of Manning's n-values in Chow (1959) as a guide, Table 2 was developed to provide general guidance in assigning n-values to this numerical model. The result is shown in Table 3. A first approximation was made for overbank n-values, and channel n-values were established by making trial runs with HEC-2 until the calculated water-surface profile matched high-water mark data taken during the 1977 flood. This was a 94,000-cfs calibration. It appeared that the initial overbank values were too low, and all overbank n-values were increased by 0.01 with the exception of those in the Williamson CBD. These were held at 0.020. Channel n-values were reestablished as described above, resulting in a more reasonable relationship between channel and overbank n-values. Very minor additional changes in localized reaches produced a computed profile which matched the observed high-water marks within 1/2 ft, except at three points in the Williamson Reach where the difference was 1.0 ft.

Sensitivity of calculated profiles to the 1977 flood discharge

17. Steps were taken to improve the calibrations in those locations. The overbanks with grassy areas were initially assigned n-values from

Table 3. These were refined using Figure 2, curve C, which is for a good stand of grass 6 to 10 in. high. Channel n-values were adjusted as required to reconstitute high-water marks using 94,000 cfs. The discharge was then changed to 117,000 cfs and n-values readjusted. Table 4 contains the calibrated Manning's n-values. There was no improvement in the Williamson reach. Results are shown in Figures 3 and 4 for the 94,000- and Figures 5 and 6 for the 117,000-cfs calibrations. Note that Figures 4 and 6 are magnifications, in the Williamson reach, of the previous figures. A check for superelevation runup was made at those locations. Depending on the choice of input variables and coefficients, runup could be on the order of 0.5 to 1.0 ft. Since the high-water-mark data were collected from both sides of the stream, the tolerance between calculated and observed water-surface elevations was relaxed from our usual 0.5 ft to a value of 1.0 ft.

18. At this point, reconstitution of the Williamson stage-discharge rating curve was attempted in order to compare a wider range of discharges with measured data. Figure 7 shows the rating curves developed by the USGS at the Williamson gage. The lower curve was developed from measured discharges up to 22,000 cfs, most of which occurred in the winter. The upper curve was developed after the 1984 flood. It is drawn through the measured 1984 data then forced through the estimated 1977 flood discharge of 94,000 cfs. Both curves display a rapid decrease in slope for gage heights above 45 ft. A variety of conditions can produce such a shape: (a) if conveyance increases rapidly for the same incremental increase in water depth, (b) if critical flow is approached, (c) if the bed is eroded, (d) if a significant reduction in roughness occurs, and (e) if a change in other hydraulic losses occurs. Discharges ranging from the 2-year event to the 100-year event were analyzed in the numerical model. Manning's n-values from the 1984 flood calibration were used for discharges less than the 1984 flood; Manning's n-values from the 1977 flood calibrations were used for discharges greater than the 1977 flood.

19. The HEC-2 results using both the 94,000- and 117,000-cfs calibration through the Williamson reach produced calculated stages which plotted reasonably well on the Williamson gage relationship except for the 1984 flood and the SPF. The SPF results, for both calibrations, showed a reversal in curvature at the upper end of the rating curve which produced an increasing slope for increasing discharge. Such a reversal does not occur in natural conditions and no hydraulic anomaly exists in the reach of the

Williamson gage. Plotting the calibrated Manning's "n" versus discharge and versus elevation showed that the "n" required to reconstitute the 1977 flood elevation was not consistent with the n-value trends displayed by the calibration of the lower discharges to the rating curve. Consequently, this analysis produced two conclusions and raised two very significant questions. The first conclusion was that Froude numbers at the Williamson gage were well below one, which indicates critical flow did not occur. The second was that bed erosion did not appear to be responsible since, during the 1984 flood, the USGS measured the channel bottom elevation which revealed an average change of less than 1 ft during the event. The questions were (a) what caused the calculated reversal in curvature and (b) how can the small channel n-values, required to reconstitute the observed high-water marks for both the 1984 and 1977 floods be justified? The search for answers started with hydrology and ended with a hydraulic analysis of flow distribution between channel and overbanks with seasonal variations in vegetative roughness.

Sensitivity of calculated results
to main stem and tributary inflows

20. After attempts to rectify this anomaly by reasonable adjustments to the model had no significant effects, attention turned to hydrologic effects. Perhaps tributary runoff during the 1984 flood in the reaches downstream of Williamson was not consistent with that produced by the unit hydrographs developed for the SPF and other hypothetical floods. The published discharge for the 1984 flood is 82,000 cfs at the Kermit gage. That is greater than the 50-year peak frequency discharge developed by ORH. However, the published discharge for the 1984 flood at the Williamson gage, 50,000 cfs, is only slightly higher than the 20-year peak frequency discharge developed by ORH.

21. Consequently, the Hydrology Section in ORH investigated the possibility and determined that the main stem discharge of 82,000 cfs at Kermit appeared to be too high. Available rainfall data for the 1984 flood were processed through their HEC-1 model which produced a discharge of 58,000 cfs. The drainage area ratio method was applied to develop intermediate flow data between Kermit and Williamson and the resulting data were inserted into the model. Slight increases in channel n-values were required to recalibrate to the 1984 high-water-mark data. Precision in the Williamson reach was not affected nor was the peculiar curvature of the Williamson stage-discharge curve resolved.

Sensitivity of calculated results to overbank flow at Williamson

22. The USGS made detailed discharge measurements at the Williamson gage during the 1984 flood as shown in Figure 8. All flow in these measurements passed under the Harvey Street Bridge. Even allowing for a looping effect, a smooth curve through the measured points indicates that the maximum discharge under the bridge was about 45,000 cfs which is 5,000 cfs less than the USGS estimate. Note that the bend in the rating curve occurs near the top of the existing floodwall, which is the elevation at which floodwaters would begin to flow into and through Williamson CBD. This resulted in the hypothesis that the missing 5,000 cfs was overbank flow through Williamson CBD and to a lesser extent in the left overbank.

23. Several investigations were made to test that hypothesis. First, the 1984 flood velocity measurements were used to develop channel and overbank roughness k_s values for the maximum discharge measurement of 42,000 cfs. With those k_s values held constant, and with the observed gage elevation for the 1977 flood, a composite k_s was calculated. Using the slope of the 1977 high-water marks and that composite k_s, the total discharge was calculated to be 62,000 cfs, which is well below the USGS value of 94,000 cfs. That flood was known to have substantial overbank flow.

24. Second, the aerial photographs taken during the 1977 flood were studied to determine the orientation of streets with respect to the valley alignment and river planform immediately upstream. Ground photographs, taken during the flood, showed wake zones at utility poles and buildings, indicating considerable flow through Williamson. It was determined that significant overbank flow could have occurred during the 1977 flood.

25. The third step was to estimate the incremental discharge between the 1984 gage height and the 1977 gage height. The USGS measured water velocities up to 8 ft/sec near midstream during the 1984 flood. Assuming a conservatively high value of 10 ft/sec and multiplying by the incremental area, i.e. the width of the Harvey Street Bridge times the elevation difference between the 1984 and 1977 flood peaks, an incremental discharge of 24,000 cfs was obtained. This, when added to the estimated 1984 channel discharge of 45,000 cfs, produces a channel discharge of 69,000 cfs, as compared with the published discharge of 94,000 cfs. This analysis indicated discharges through Williamson CBD of 5,000 cfs and 25,000 cfs for the 1984 and 1977 floods, respectively.

26. The next step was to refine the HEC-2 numerical model to reflect the geometry and hydraulic roughness of buildings and streets in Williamson. Chow (1959) suggests n-values ranging from 0.010 to 0.015 for concrete or asphalt surfaces. A value of 0.02 was considered more appropriate due to turbulence between and around buildings, parked cars, and other flow obstructions. The 1984 flood was analyzed first, and the HEC-2 model calculated 45,000 cfs in the channel and 5,000 cfs through Williamson which agrees with the rating curve loop analysis. Results of the 1977 flood analysis showed channel and overbank discharges of 64,000 cfs and 29,000 cfs (1,000 cfs in left overbank), respectively. Those results agree with the estimates above from the rating curve analyses, 62,000 and 69,000 cfs for the channel discharge and 32,000 and 25,000 cfs through Williamson. Consequently, the right overbank in Williamson had sufficient conveyance to transport considerable flows.

27. The next question addressed in evaluating this hypothesis was whether or not the water could flow over the existing floodwall at the discharge rate to supply that overbank conveyance. To check this, the HEC-2 model was set up for the split flow option which simulates spatially varied, lateral outflow over weirs and levees. Using a weir length of the floodwall, adjusted for adjacent buildings, and submerged-flow weir coefficients of 1.5 to 1.8, a potential lateral outflow from the channel of 48,000 cfs was calculated for the 1977 flood discharge of 94,000 cfs. This outflow potential is greater than the conveyance through Williamson, indicating that the estimated right overbank discharge of 29,000 cfs for the 1977 flood could have been reached. This approach was then used for the 1984 flood resulting in a channel outflow of 5,000 cfs, again indicating that the conveyance potential could have been reached. Consequently, the significant overbank flow in Williamson appears to be reasonable and accounts for much of the shape of the curve. Finally, the bend in the curve is influenced by the height of tree roughness on the channel banks. That corresponds to the reduction in roughness impact on rating curve shape. Once trees are completely submerged, relative roughness begins to decrease significantly.

Sensitivity of calculated results to seasonal variations in vegetative roughness

28. The final effort in explaining rating curve shape involves seasonal differences in vegetative roughness. As shown in Figure 7, the 1984 flood

plots about 3 ft above the pre-1984 curve, causing a sharp bend in curvature in USGS curve No. 13. That flood occurred in May, a time when there was considerable foliage on the trees and bushes, which are largely conifers. The pre-1984 rating curve was extrapolated from measured discharges up to 22,000 cfs and forced through the indirect measurement of the 1977 event, which occurred in April (minimal foliage). The 1984 flood was the highest measured discharge. If the 1977 flood had occurred later in the spring, it could be argued that the water-surface elevation would have been a few feet higher as evidenced by the 1984 flood elevation being above the pre-1984 curve. Conversely, it could be argued that if the 1984 flood occurred in the winter, the flood elevation would have been lower, perhaps even on curve No. 8. The final step in the rating curve analysis was to recognize the possibility of the SPF occurring at any time of the year, and establish two rating curves at Williamson, spring and winter curves, and then extrapolate both to the SPF discharge. The USGS curve No. 8 was adopted as the winter condition and an upper curve calculated for the spring condition. The results are shown in Figure 9 for a 94,000-cfs calibration and in Figure 10 for the 117,000-cfs calibration. Details of the extrapolation procedure are discussed later.

Summary of HEC-2 Model and Hydraulic Parameter Evaluations

29. The works described thus far addressed tasks a and b of the study objectives that required refinement of the HEC-2 numerical model and identification of hydraulic parameters which significantly influenced the water-surface profiles. Refinements to the HEC-2 model included substituting field data at bridges, developing reach lengths, and assugning n-values by vegetation and land use. Channel limits were also reestablished to better approximate the limits of bank vegetation. For task b, sensitivity of the calculated profiles was evaluated to determine the significant hydraulic parameters. Superelevation, bed scour during floods, local inflows, overbank flows, relative roughness, and seasonal vegetation roughness were analyzed. The two key sources of field data for these studies were high-water marks during the 1984 and 1977 floods and the USGS gage records at Williamson. Bed scour during events was found to be negligible. Superelevation did not impact on the result except as it indicated we should relax the calibration tolerance

from a 1/2 ft to 1 ft. Local inflow changes improved agreement between calculated and observed profiles between the gages. The identification of significant overbank flow through the town of Williamson, changes in relative roughness as the rare floods overtopped all trees, and seasonal changes in vegetative roughness were the three most significant hydraulic parameters. So significant is the maximum discharge during the 1977 flood event that two extrapolations were made, one for a 94,000-cfs event and the other for a 117,000-cfs event.

30. Because of the importance of hydraulic roughness, the procedure developed for extrapolating the rating curves to the SPF is presented in detail in the following paragraphs. It follows EM 1110-2-1601 (OCE 1970), which recognizes the importance of "relative roughness," and uses observed data to calculate roughness height, k_s, which is then used to calculate relative roughness at SPF depths.

Extrapolation of the Stage-Discharge Curve for Existing Conditions

The general approach using the winter rating curve

31. As previously stated, USGS rating curve No. 8, Figure 7, was adopted as the winter condition. For a range of discharges up to the 1977 flood, the channel n-values in the HEC-2 numerical model were adjusted, by successive approximations, to reproduce the rating curve at the Williamson gage. The resulting hydraulic parameters were used to compute a composite channel roughness/ element height k_s for each elevation. An example is shown in Figure 11 and the computation is explained in Appendix A.

32. The minimum possible composite k_s is the streambed roughness associated with "grain roughness." Based on samples taken during a low-flow period at the Harvey Street Bridge, a D_{84} of 10 mm and a D_{50} of 2 mm were estimated. Using Limerino's (1970) relationship, Manning's n-value was calculated to be 0.022. From Strickler's (1923) relationship, an n-value of 0.015 was calculated. Using a representative n-value of 0.02 and the relationship of Manning's equation to Chezy C and roughness k_s from Plate 3 of EM 1110-2-1601, a k_s value of 0.07 was calculated. These calculations are in Appendix B. The k_s versus elevation curve, Figure 11, was extrapolated asymptotically toward that k_s value of 0.07 at a high elevation.

33. A trial elevation was estimated for the SPF by extrapolating the rating curve and that elevation was used to estimate a SPF relative roughness which was then converted to an equivalent channel n-value using Plate 3 of the EM. That was input to HEC-2 and the SPF profile calculated. The HEC-2 run produced an elevation that was then checked against the estimated SPF elevation. If the estimated and computed elevations differed, successive approximations were made until the difference was negligible. The 94,000 cfs-calibration channel discharge versus composite channel k_s curve is shown in Figure 12.

34. These figures show that k_s increases for increasing elevation, or for increasing discharge, up to a maximum and then decreases as the elevation/discharge continues to increase. Generally, k_s should be a constant because channel bed irregularities and grain roughness are fairly constant. That is applicable only if k_s is the same along the entire cross section and if trees are not present in the section. EM 1110-2-1601 shows a procedure to develop an alpha-weighted, composite k_s for cross sections with subsections having different k_s values. For discussion sake, assume that k_s for each subsection stays constant. A typical cross section on Tug Fork consists of trees and undergrowth on the banks and an alluvial streambed. At low stages, all of the water is completely in the streambed, which has a relatively low k_s as compared with the banks. As the water level rises, the composite roughness is increased by additional roughness from the growth on the banks. This continues until the trees are overtopped. As the water-surface elevation continues to rise above the trees, the conveyance in the streambed increases faster than the conveyance of the banks. This results in a decreasing composite k_s for increasing discharge or elevation. Theoretically, at a very high water depth, the bank conveyance becomes negligible as compared with the channel conveyance. This results in a composite k_s approaching that of the streambed k_s . Appendix B contains sample calculations of a simplified cross section typical of Tug Fork using a channel k_s determined from low flows and bank k_s determined from measurements of the 1984 flood. The results in Appendix C are similar to those shown in Figures 11 and 12.

35. Using the relationship between the Chezy and Manning equations, the composite k_s can be converted to Manning's n-values versus elevation, Figure 13. A more familiar plot is n-value versus discharge, Figure 14. Using that graph of n-values, HEC-2 calculated the water-surface elevations

shown in Figure 9, which is the extrapolated stage-discharge curve for existing conditions for the 94,000-cfs calibration.

Development of the spring rating curve

36. The only reliable data for a spring flood were the 1984 data. Calibration to these data produced a composite channel k_s that was plotted with the winter elevation versus composite k_s on Figure 11. The extrapolation was accomplished by going through the 1984 point and following the general curvature of the winter curve. The SPF elevation was determined using the same procedure as used in the winter rating curve extrapolation.

Winter and spring rating curves for 117,000-cfs calibration

37. Because of the uncertainty of the maximum discharge for the 1977 flood, the procedure described above was repeated for the 117,000-cfs calibration resulting in Figures 10, 15, 16, 17, and 18.

SPF water-surface profiles

38. Using the extrapolated n-values, the SPF water-surface profile for existing conditions was calculated based on both the 94,000- and the 117,000-cfs calibrations. These profiles are plotted in Figures 19 and 20.

Project Conditions Analysis

Proposed project

39. The West Williamson floodwall project limits are from river mile (R.M.) 55.35 to R.M. 56.55. The floodwall is located on the right descending bank of the Tug Fork. The riverbank outside the floodwall will be cleared and graded to a slope of 2.5H on 1V with a 30-ft-wide bench between the top of slope and the wall. The bench elevation varies from el 660 to el 665 in the upstream direction. There is a channel modification (straightening) from R.M. 55.6 to 55.8. Stone slope protection (SSP) will be placed from streambed to el 648 on the right bank along the floodwall limits. At the channel modification, SSP will be placed on both banks of the Tug Fork.

40. The South Williamson floodwall project limits are from R.M. 56.45 to R.M. 56.95. The floodwall ties into the US 119 Highway fill at the new US 119 bridge on the upstream end. The floodwall is located on the left descending bank of the Tug Fork. The riverbank outside the floodwall will be cleared and graded to a slope of 2.5H on 1V with a 30-ft-wide bench between

the top of slope and the wall. The bench elevation varies from el 662 to el 664 in the upstream direction. SSP will be placed from streambed to el 648 on the left bank along the floodwall limits.

41. The Williamson CBD floodwall (sheet-pile cells) project limits are from R.M. 57.0 to R.M. 57.6. The floodwall is located on the right descending bank of the Tug Fork. The riverbank outside the floodwall, starting just upstream of the Harvey Street Bridge, will be cleared and graded to a slope of 2.5H on 1V with a 10- to 20-ft-wide bench between the top of slope and the wall. The bench will be at el 640. SSP will be placed from streambed to el 651 or top of bench, whichever is lower, on the right bank from the Harvey Street pumping station (R.M. 57.2) upstream to the Williamson Creek pumping station outfall (R.M. 57.6).

HEC-2 modifications for project conditions

42. The floodwalls and associated bank changes described above were all coded in the HEC-2 backwater model using GR points. The floodwalls in particular were coded by increasing the GR station 1 ft from the toe of wall elevation to top of the wall elevation. The tops of walls were set sufficiently high in elevation to prevent overtopping and flow landward of the walls.

43. The overbank Mannings n values were the same as in the calibration phase except where the proposed floodwalls were located. These floodwalls were assigned an n-value of 0.02 rather than 0.012 to 0.015 as suggested by Chow (1959) because of the "cellular" design of the floodwalls.

44. Under project conditions, all the flow at Williamson is to be confined in the channel. Since the elevation versus composite k_s curve was developed with some of the discharge in the overbank, it is not valid for project conditions because for the same water-surface elevation, the energy slope and discharge are different. However, the discharge versus composite k_s is still valid if the channel discharge is used to determine the k_s. The energy slopes of the project conditions are also very similar to the existing conditions. Because of this, the discharge versus composite k_s relationship was used to determine the k_s for project conditions discharges, which was then used to produce project conditions Mannings n.

45. When the flow is confined by floodwalls, a shear force is exerted on the streambed by these lateral boundaries. If significant, this shear force could increase the overall roughness of the streambed. Application of

the sidewall roughness correction procedure proposed by Vanoni and Brooks (1957) showed that no such correction was needed.

Rating curves for project conditions

46. The composite k_s extrapolations were used to determine the Mannings n-values that were input to the HEC-2 model and used to calculate project conditions winter and spring rating curves at Williamson for the 94,000-cfs calibration, Figure 21. The same relationships for the 117,000-cfs calibration are shown in Figure 22.

Project conditions SPF water-surface profiles

47. The calculated, SPF profiles are shown in Figures 23 and 24. Table 5 contains the calibrated Manning's n values used in the modified reaches as well as the reference n-values in those reaches for existing conditions.

Kermit gage sensitivity

48. In view of the uncertainties in rating curve extrapolations, the sensitivity of the SPF water-surface profiles in Williamson to variations in the Kermit gage starting water-surface elevation was examined. Variations of ±2 ft at the Kermit gage produced changes of only hundredths of a foot at Williamson.

Summary

49. The Huntington District (ORH) requested aid in performing a hydraulic study of the Tug Fork River, West Virginia, from the Kermit gage to the town of Williamson. This area is the site of a major flood protection project and is to be protected to the Standard Project Flood (SPF). The objectives of the study were to aid in updating and calibrating ORH's existing HEC-2 model, identify and incorporate into the model important hydraulic parameters, develop a procedure for extrapolation of the rating curve at Williamson, and determine the SPF water-surface profile under project conditions.

Original ORH HEC-2 model

50. The bulk of the HEC-2 geometric input data was obtained from orthophoto mapping at contour intervals of 5 ft. Field surveys were conducted to obtain bridge and stream bottom data. HEC-2 calibration to the 1977 flood

discharge of 94,000 cfs resulted in a channel Manning's "n" of 0.040 with overbank n-values of 0.060 to 0.067, which were used for the entire study reach. Extrapolation to the SPF resulted in a channel Manning's "n" of 0.024 with overbank n-values of 0.036 to 0.040. Expansion and contraction coefficients of 0.3 and 0.1, respectively, were used for all discharges.

Geometric adjustments to the HEC-2 model

51. Using mylar overlays of the mapping as guides, overbank reach lengths were adjusted to reflect the length of the centroid of the flow lines. Representations of buildings were adjusted as needed and bridge survey data were used to update the geometry. Geometric adjustments were also made in the town of Williamson to allow flow through the central business district (CBD).

Estimation of the 1977 flood discharge

52. ORH, using unit hydrograph and routing techniques, obtained a discharge of 117,000 cfs for the 1977 flood which differed from the USGS estimate of 94,000 cfs. Since the use of either one would affect the SPF extrapolation, different calibrations were performed based upon the two discharges.

HEC-2 calibration for the 1984 and 1977 floods

53. Using Chow (1959) as a guide, Manning's n-values were assigned to specific reaches of the river and put in the HEC-2 model. The initial n-values were adjusted to achieve observed high-water marks. These marks were reproduced within ±0.5 ft except for three marks that were reproduced within ±1.0 ft which was attributable to superelevation runup at bends.

54. Due to inconsistencies in the water-surface profiles for the 1984 flood, adjustments to the initial tributary discharges were made after the rainfall data were reexamined and the 1984 flood reconstituted. This changed the main stem discharge at the Kermit gage from 82,000 to 58,000 cfs for the 1984 flood.

55. The calibration to the 1984 flood resulted in a channel Manning's "n" of 0.058 at the USGS gage in Williamson. The 1977 flood calibration produced channel n-values of 0.041 and 0.028 for the 94,000- and 117,000-cfs calibrations, respectively.

56. Analyses of the detailed USGS discharge/velocity measurements from the 1984 flood indicated that significant flow through the Williamson CBD

occurred during the 1977 flood. To simulate this, the HEC-2 model was adjusted to reflect the geometry of the buildings and streets and this overbank area was assigned a Manning's n-value of 0.020. Checks were made to assure that sideflow over the existing floodwall was sufficient to meet the CBD conveyance potential.

Analysis of rating curve at Williamson

57. Analysis of the 1984 and 1977 floods and pre-1984 USGS rating curves suggests that discharges under winter or spring foliage conditions would have different water-surface elevations. This led to the adoption of winter and spring rating curves with the pre-1984 USGS rating curve (up to the 1977 flood discharge) as the winter condition and with the spring curve based upon the 1984 flood measurements. Extrapolation of the rating curves to the SPF was accomplished by first plotting channel composite roughness, k_s, versus water-surface elevation. Knowing the k_s and hydraulic radius for an estimated elevation, a Manning's "n" was calculated and input to the HEC-2 model. The resulting water-surface elevation was then checked against the estimated elevation. This procedure was repeated until the estimated and HEC-2 water-surface elevation matched. For the 94,000-cfs calibration, the extrapolated SPF channel Manning's n-values at Williamson for the winter and spring conditions are 0.034 and 0.038, respectively. The 117,000-cfs calibration results for the winter and spring conditions are 0.024 and 0.026, respectively.

Project conditions analysis

58. The HEC-2 model was modified for project conditions by adjusting the geometry and roughness associated with floodwalls and channel modifications. The SPF Manning's "n" under project conditions was determined using the same procedure as was used in the existing conditions analysis except the composite k_s versus channel discharge relationship was used. The n-values differ because under project conditions, almost all the flow is confined in the channel. For the project conditions 94,000-cfs calibration, the SPF Manning's n-values at Williamson for the winter and spring conditions are 0.030 and 0.032, respectively. The 117,000 cfs calibration results for the winter and spring conditions are 0.023 and 0.024, respectively.

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Table 1

Original HEC-2 Input

Tug Fork River from Kermit to Williamson, W. Va.

Flood	Total Discharge cfs	Mannings "n" Channel	Overbank
2-year	22,000	0.052	0.078 - 0.087
5-year	31,000	0.052	0.078 - 0.087
10-year	38,300	0.051	0.077 - 0.085
20-year	47,100	0.049	0.073 - 0.082
50-year	58,600	0.046	0.069 - 0.077
100-year	68,000	0.043	0.064 - 0.072
200-year	78,100	0.043	0.064 - 0.072
April 1977	94,000	0.040	0.060 - 0.067
SPF	167,000	0.024	0.036 - 0.040

Table 2

General Roughness Descriptions and Assignments of Manning's n

OVERBANKS

Overbank Description	Range of n
Clear - Grass	0.035 - 0.045
Light - Medium Brush	0.050 - 0.060
Heavy Brush - Trees	0.070 - 0.080
Development - streets and buildings not aligned with flow lines	0.050 - 0.060
Development - streets and buildings aligned with flow but no easy access	0.040 - 0.050
Development - streets and buildings aligned with flow and easy access	0.020

CHANNEL

Channel Area and Description	n
Bottom	0.025
Sides - Light to Medium Growth	0.030 - 0.040
Sides - Medium to Dense Growth	0.040 - 0.050
At SPF level	0.040

Table 3

Roughness Description a[illegible]ing's "n" Values for 1977 Flood Calibration

Reach River Miles	Left Overbank Description	n	Channel Description*	n	Right Overbank Description	n
38.4-43.86	Clearings of Grassy and Developed Areas to Light to Medium Dense Brush	0.01 to 0.069 Avg 0.057	Gradual Bendways; Typical Side Slopes of Light to Medium Dense Brush with Reaches of Medium Dense Brush	0.036	Clearings with Scattered Development and Road and Railroad ROW's	0.041 to 0.069 Avg 0.044
43.86-49.07	Generally Clearings of Grassy and Developed Areas with Intermediate Reaches of Light to Medium Dense Brush	0.041 to 0.069 Avg 0.045	Increased Sinuosity; Sharper Bendways; Typical Side Slopes of Medium to Heavy Dense Brush with Reaches of Light to Medium Dense Brush	0.041	Clearings with Reach of Development and Scattered Vegetation; Grassy Reach and a Reach of Light to Medium Dense Brush	0.041 to 0.048 Avg 0.045
49.07-53.86	Grassy Clearings with Some Development; Reaches of Light to Medium Dense Brush	0.044 to 0.051 Avg 0.045	Gradual Bendways; Typical Side Slopes of Light to Medium Dense Brush; A Reach of Medium to Dense Brush	0.036	Developed with Short Reaches of Grassy to Light to Medium Dense Brush	0.041 to 0.048 Avg 0.045

(Continued)

Table 3 (Concluded)

Reach River Miles	Left Overbank Description	n	Channel Description	n	Right Overbank Description	n
53.86-54.16	Medium Brush and Development	0.044	Exceptionally Sharp Bend with Reverse Curvature; Medium Brush	0.046	Medium Brush	0.041
54.16-56.24	Reaches of Development (not aligned with flow lines) and of Light to Medium Brush	0.048 to 0.058 Avg 0.054	Increased Sinuosity with Sharper Bendways; Reaches of Medium to Heavy and Light to Medium Dense Brush	0.041	Heavier Development (not aligned with flow line) with Reaches of Light Brush	0.041 to 0.058 Avg 0.050
56.24-56.49	Medium Brush and Development	0.048	Exceptionally Sharp Bend with Reverse Curvature Medium Brush	0.046	Development (not aligned with flow lines) with Reaches of Light Brush	0.048
56.49-57.81	Development (not aligned with flow lines)	0.044 to 0.048 Avg 0.046	Increased Sinuosity with Sharper Bendway; Medium Dense Brush with Trees	0.041	Heavy Development (aligned with flow lines) with Cleared Reaches of Crassy to Light Brush	0.020 to 0.048 Avg 0.034

* Channel area includes the river bottom itself. Therefore, channel "n" value assignments are composite in nature and will be lower than overbank areas with a similar description.

** Discharge = USGS Q = 94,000 cfs through Williamson, W.Va., CBD reach.

Table 4

Main Channel Manning's n Values for

Historic Flood Calibrations

Reach River Miles	1984 Flood	1977 Flood Q=94,000 cfs	1977 Flood Q=117,000 cfs
38.4 - 43.86	0.051	0.036	0.025
43.86 - 49.07	0.058	0.041	0.028
49.07 - 53.86	0.051	0.036	0.024
53.86 - 54.16	0.065	0.046	0.031
54.16 - 56.24	0.058	0.041	0.028
56.24 - 56.49	0.065	0.046	0.031
56.49 - 57.81*	0.058	0.041	0.028

* Reach containing USGS gage in Williamson near Harvey Stress Bridge.

Table 5

Main Channel Manning's n Values

SPF Event (166,950 cfs), Existing Conditions

Reach River Miles	94,000-cfs Calibration Winter Channel Q = 115,000 cfs	94,000-cfs Calibration Spring Channel Q = 109,000 cfs	117,000-cfs Calibration Winter Channel Q = 132,000 cfs	117,000-cfs Calibration Spring Channel Q = 127,000 cfs
54.16 - 56.24	0.034	0.038	0.024	0.026
56.24 - 56.49	0.038	0.043	0.027	0.029
56.49 - 57.81	0.034	0.038	0.024	0.026

Main Channel Manning's n Values

SPF Event (166,950 cfs), Project Conditions

Reach River Miles	94,000-cfs Calibration Winter Channel Q = 159,000 cfs	94,000-cfs Calibration Spring Channel Q = 156,900 cfs	117,000-cfs Calibration Winter Channel Q = 162,000 cfs	117,000-cfs Calibration Spring Channel Q = 161,000 cfs
54.16 - 56.24	0.030	0.032	0.023	0.024
56.24 - 56.49	0.034	0.036	0.026	0.027
56.49 - 57.81	0.030	0.032	0.023	0.024

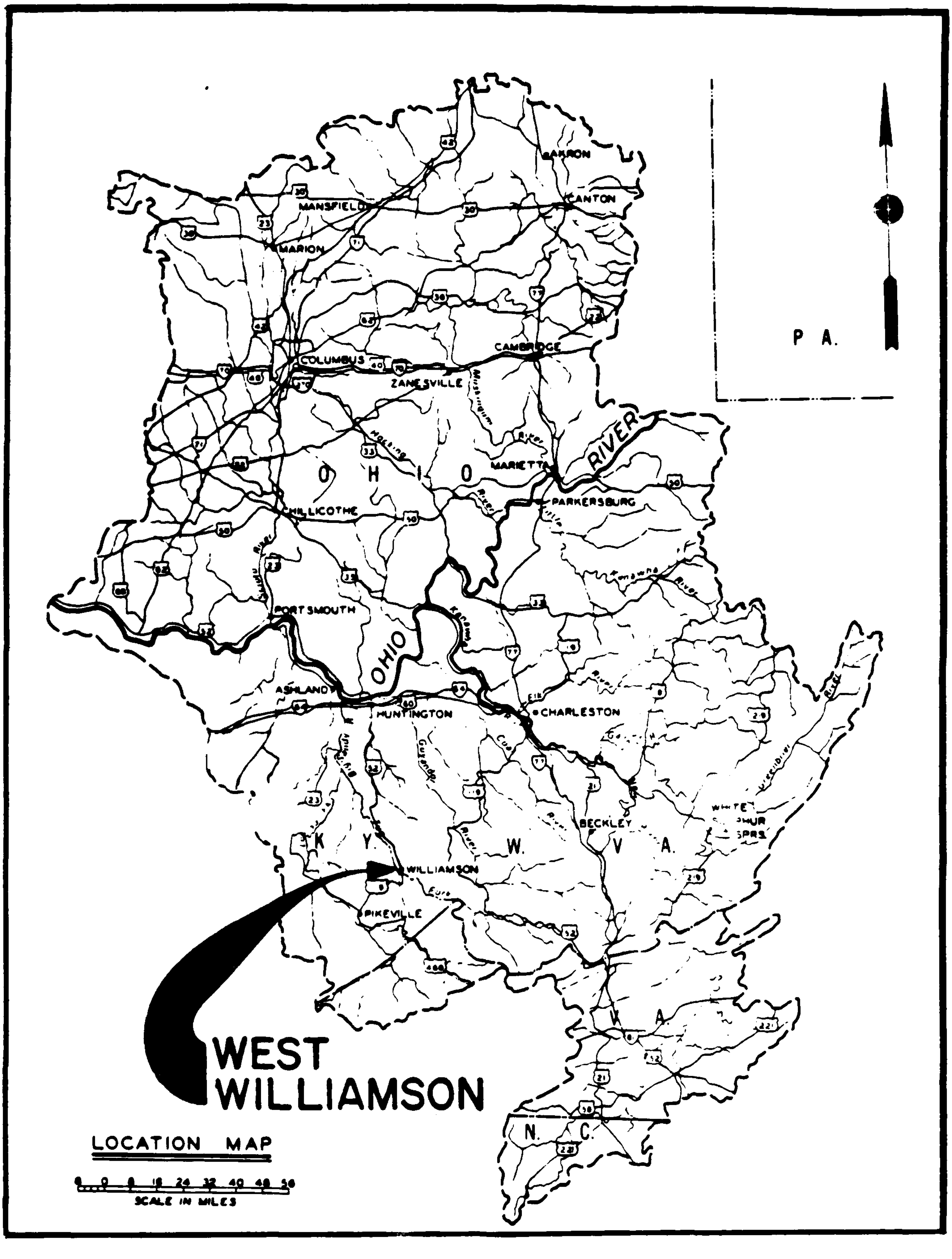

Figure 1. Location map

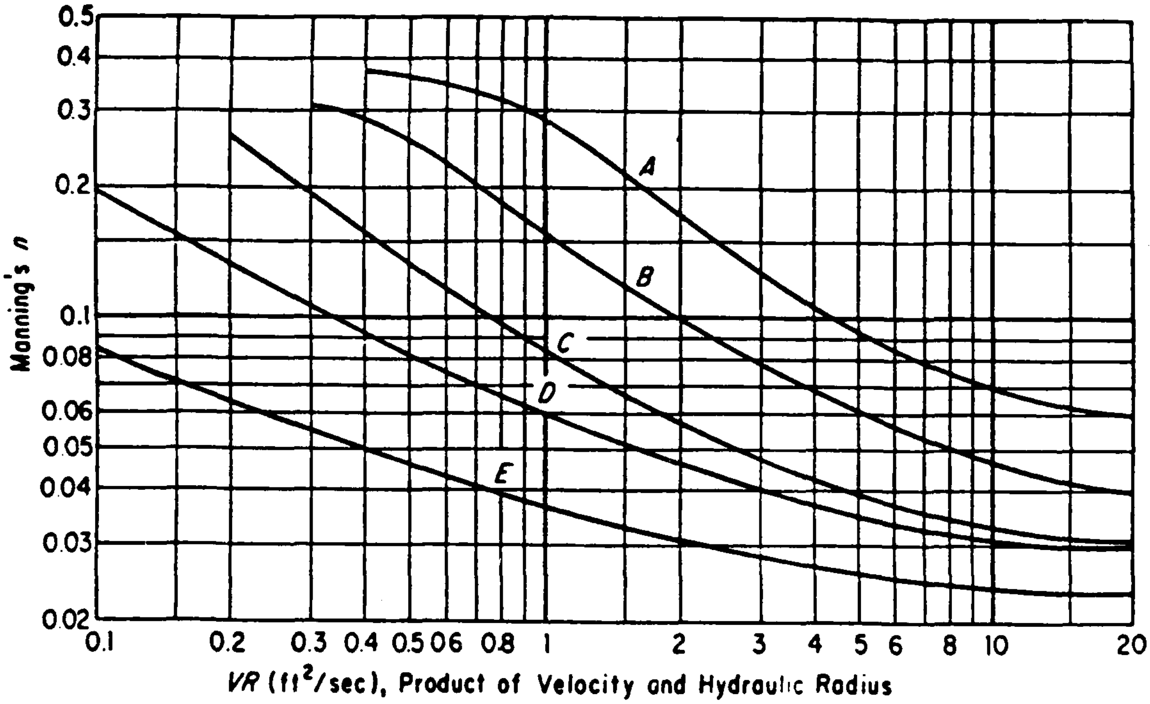

Figure 2. Overlook roughness relationship
(after Henderson 1966)

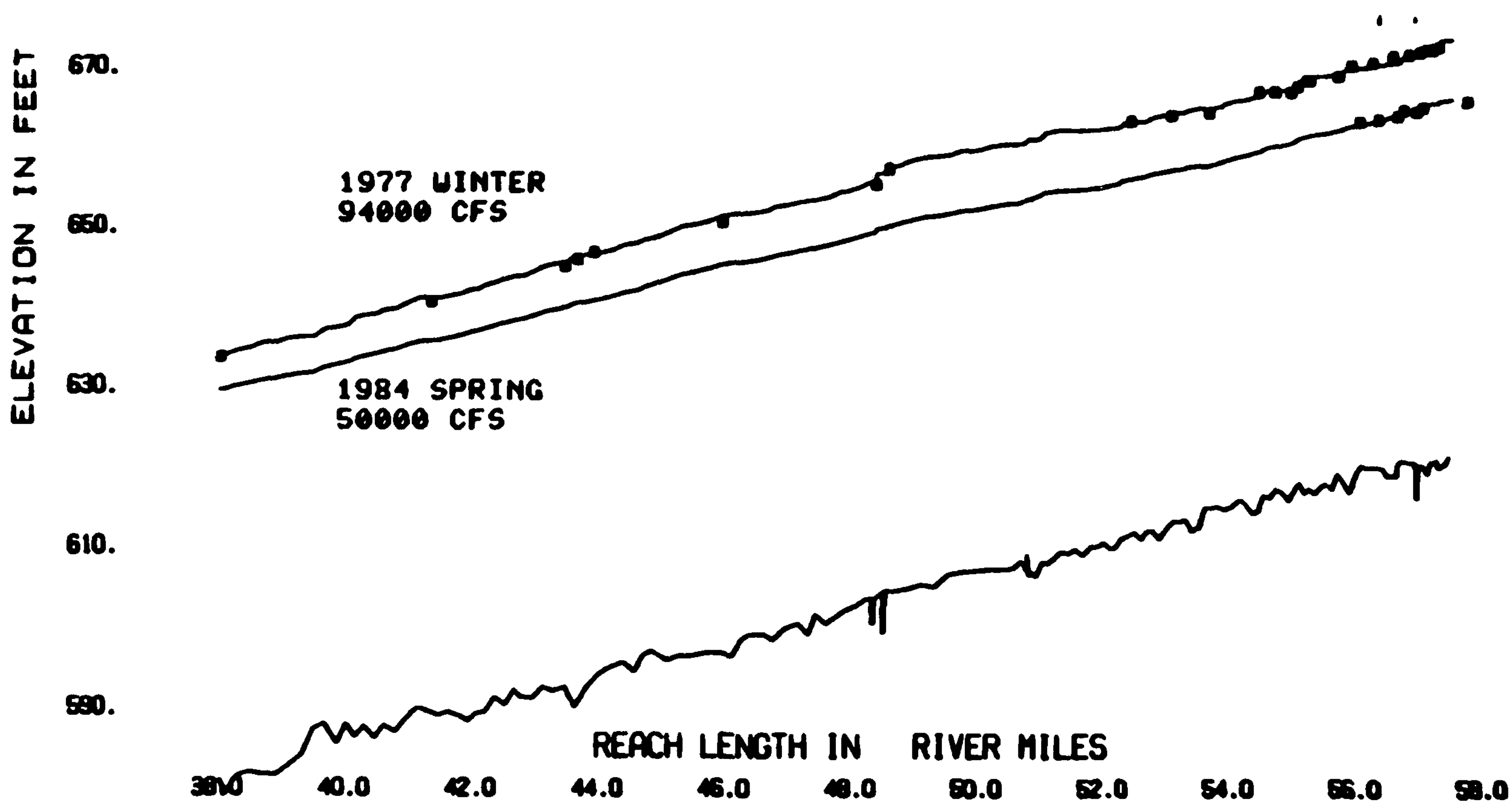

Figure 3. Calibrated HEC-2 water-surface profiles, Kermit to Williamson, W. Va., 94,000-cfs calibration

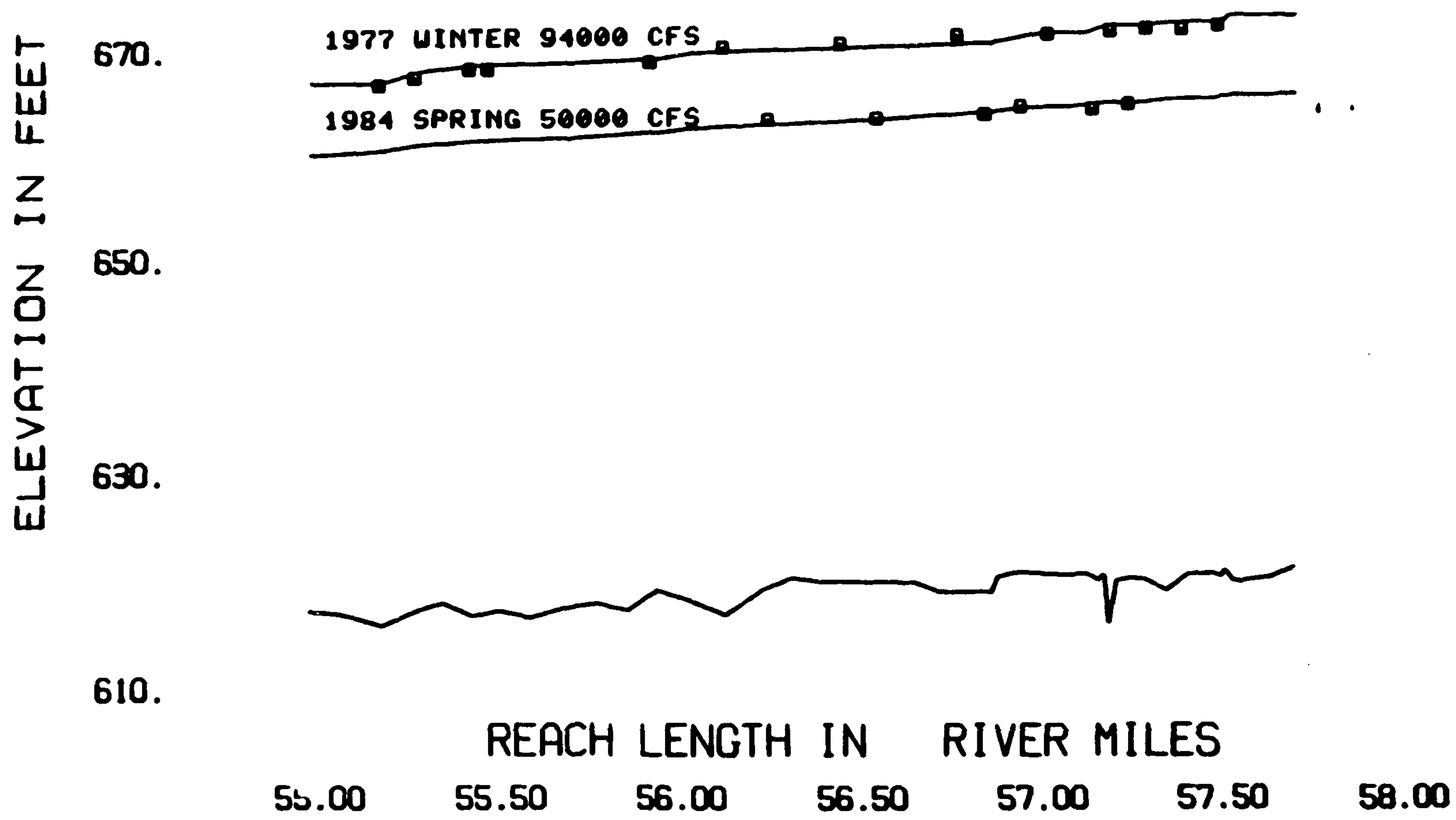

Figure 4. Calibrated HEC-2 water-surface profiles, Williamson, W. Va., area, 94,000-cfs calibration

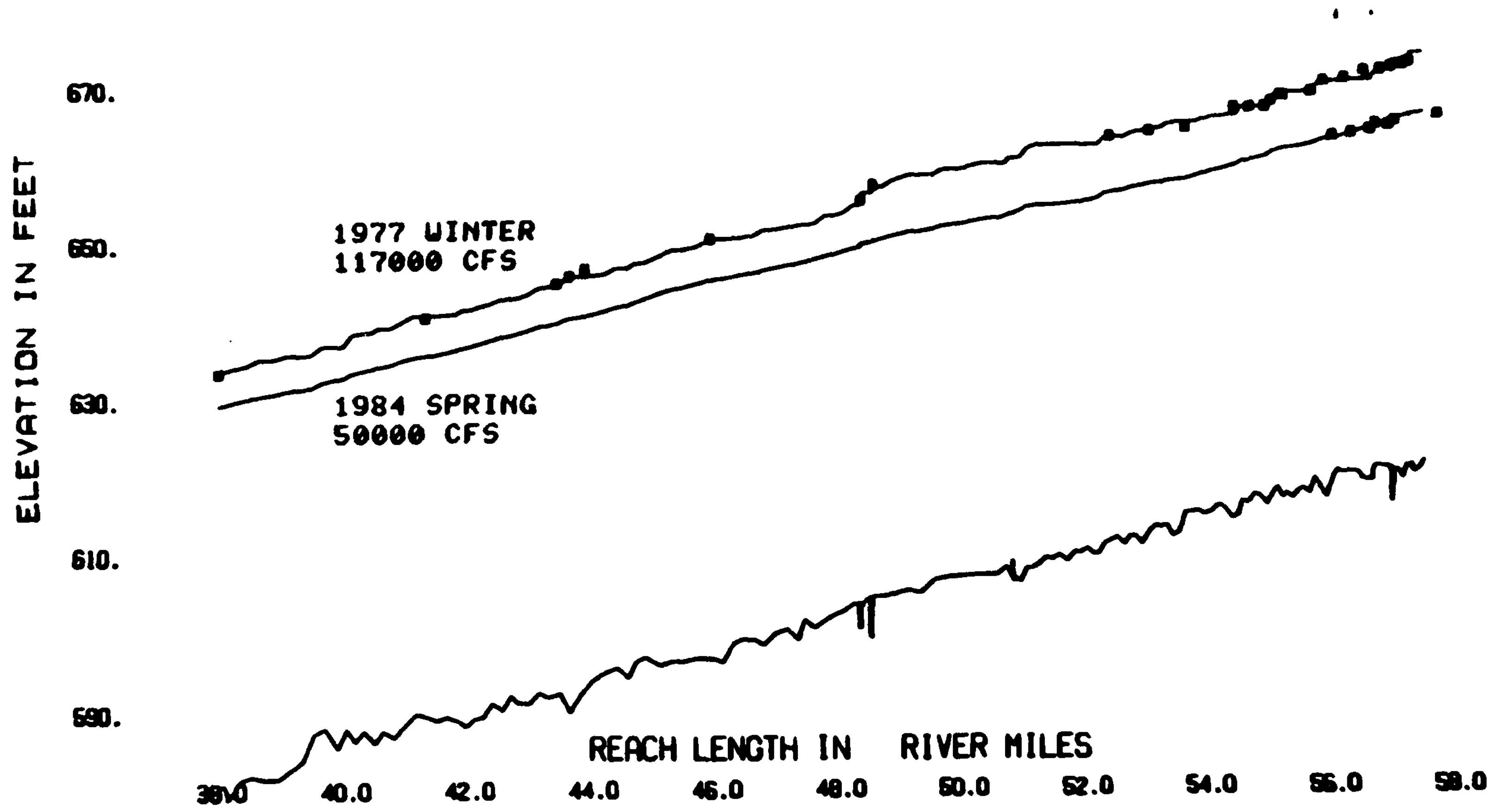

Figure 5. Calibrated HEC-2 water-surface profiles, Kermit to Williamson, W. Va., 117,000-cfs calibration

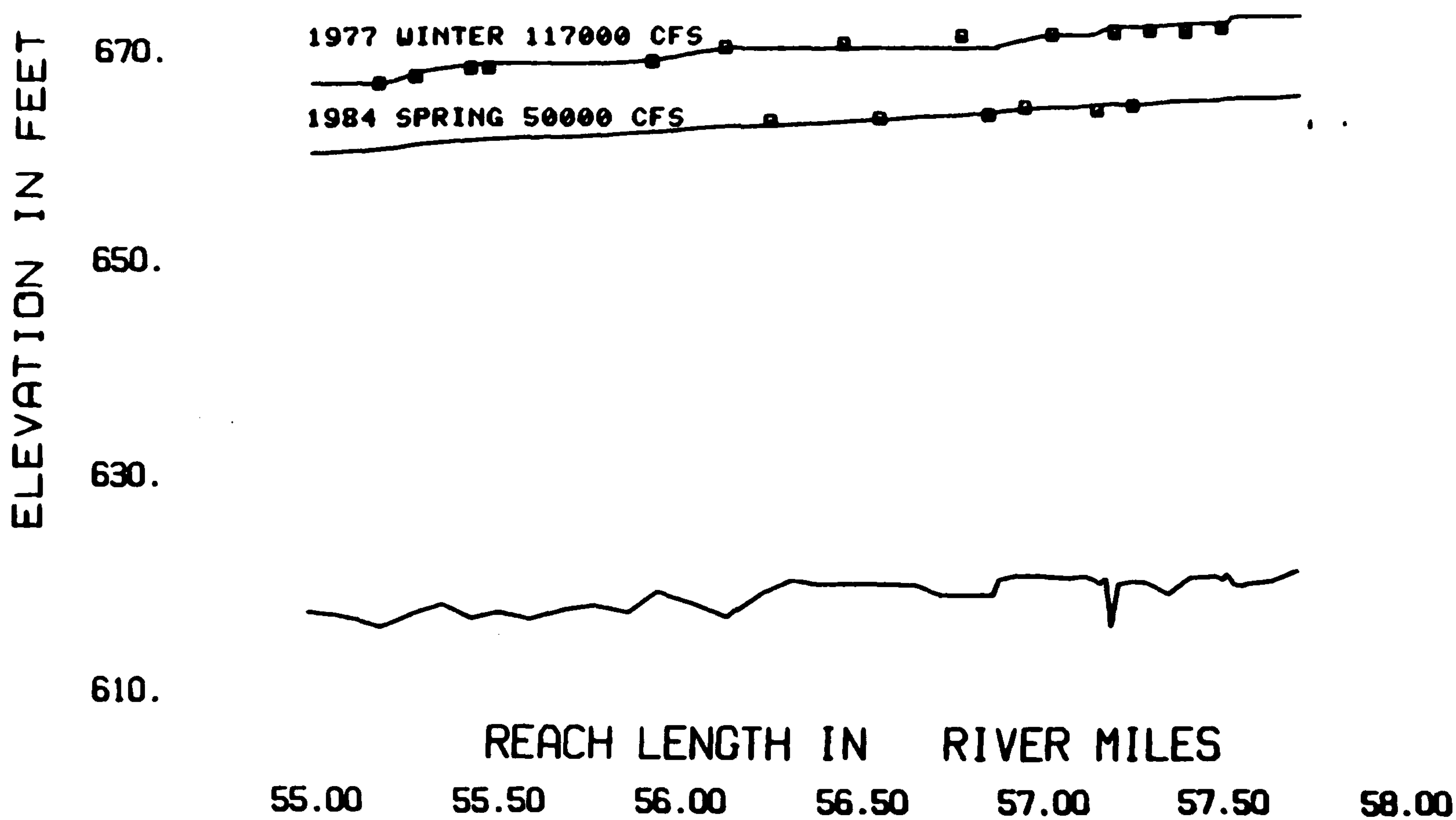

Figure 6. Calibrated HEC-2 water-surface profiles, Williamson, W. Va., area, 117,000-cfs calibration

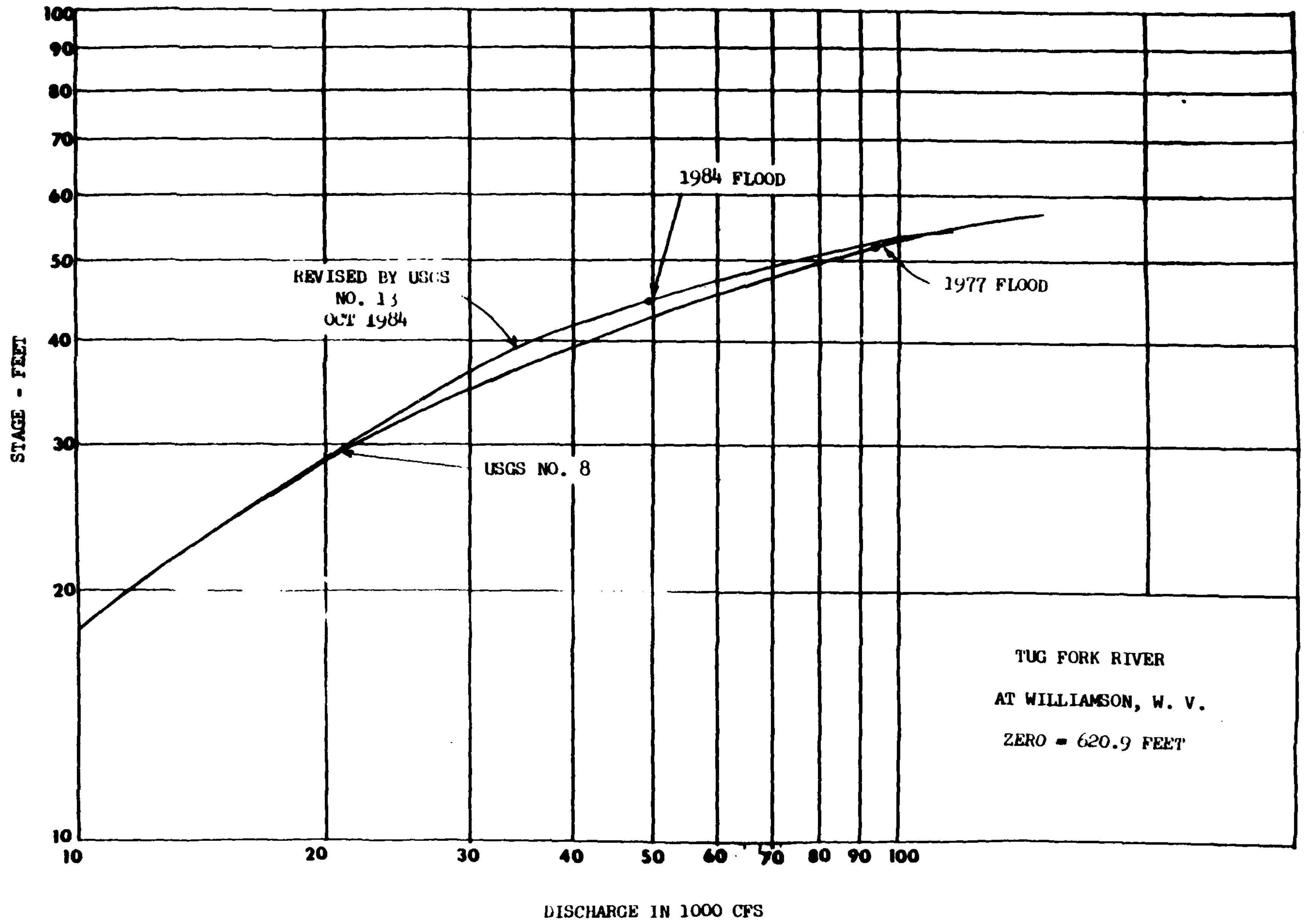

Figure 7. USGS rating curve at Williamson, W. Va., gage

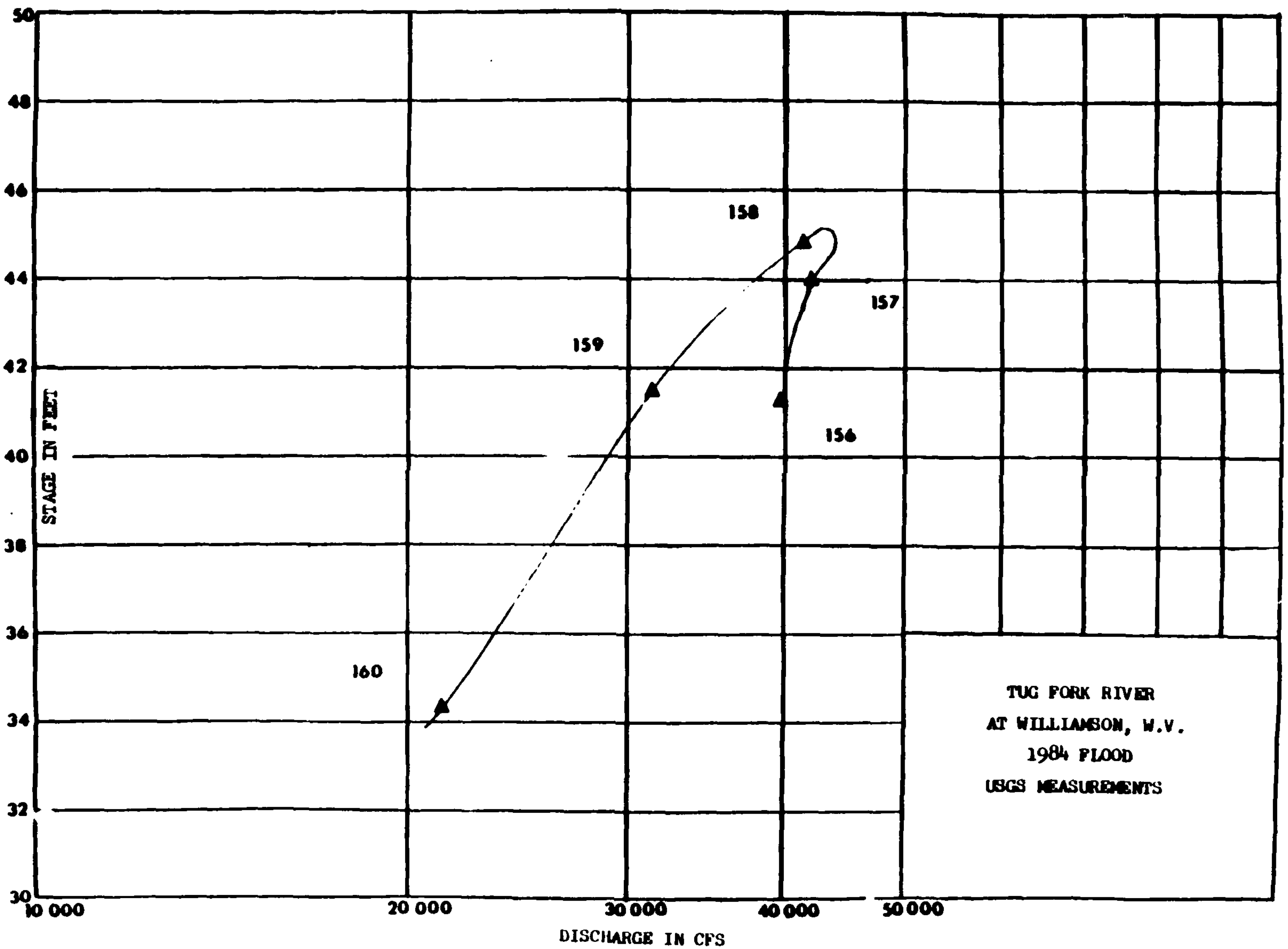

Figure 8. Loop effect for the 1984 flood at Williamson, W. Va., gage

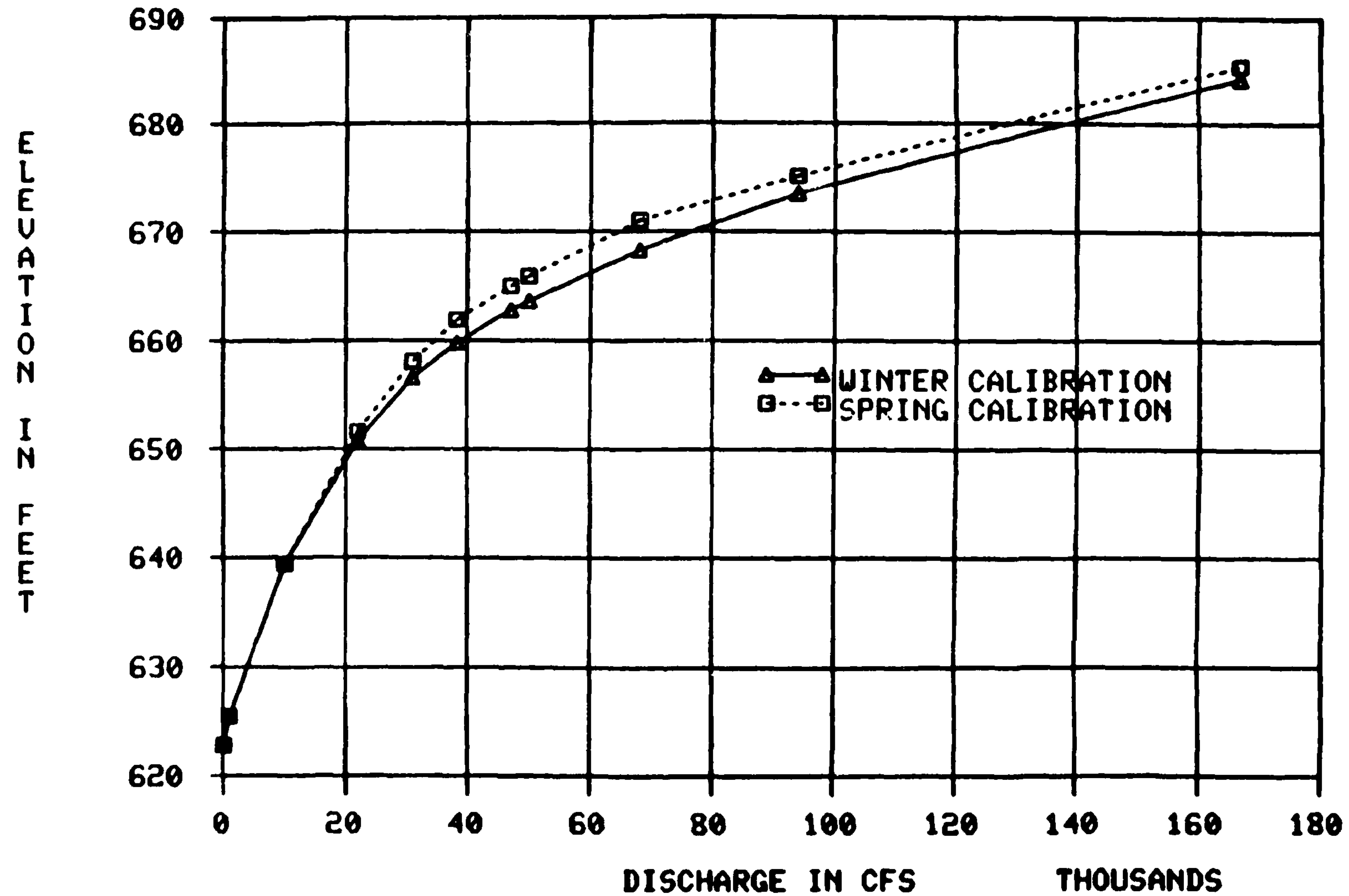

Figure 9. Existing conditions rating curve at Williamson gage, 94,000-cfs calibration

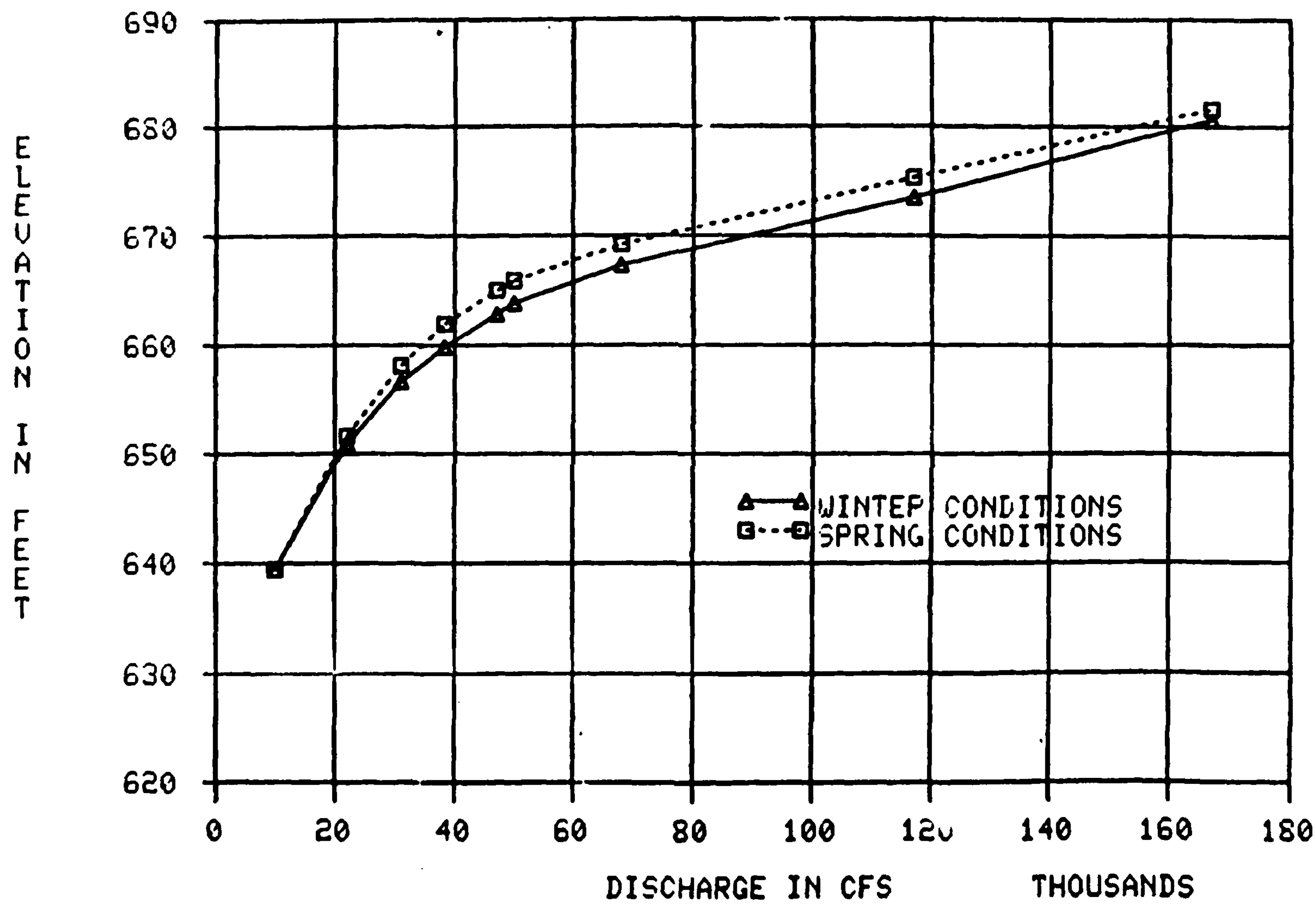

Figure 10. Existing conditions rating curve at Williamson gage, 117,000-cfs calibration

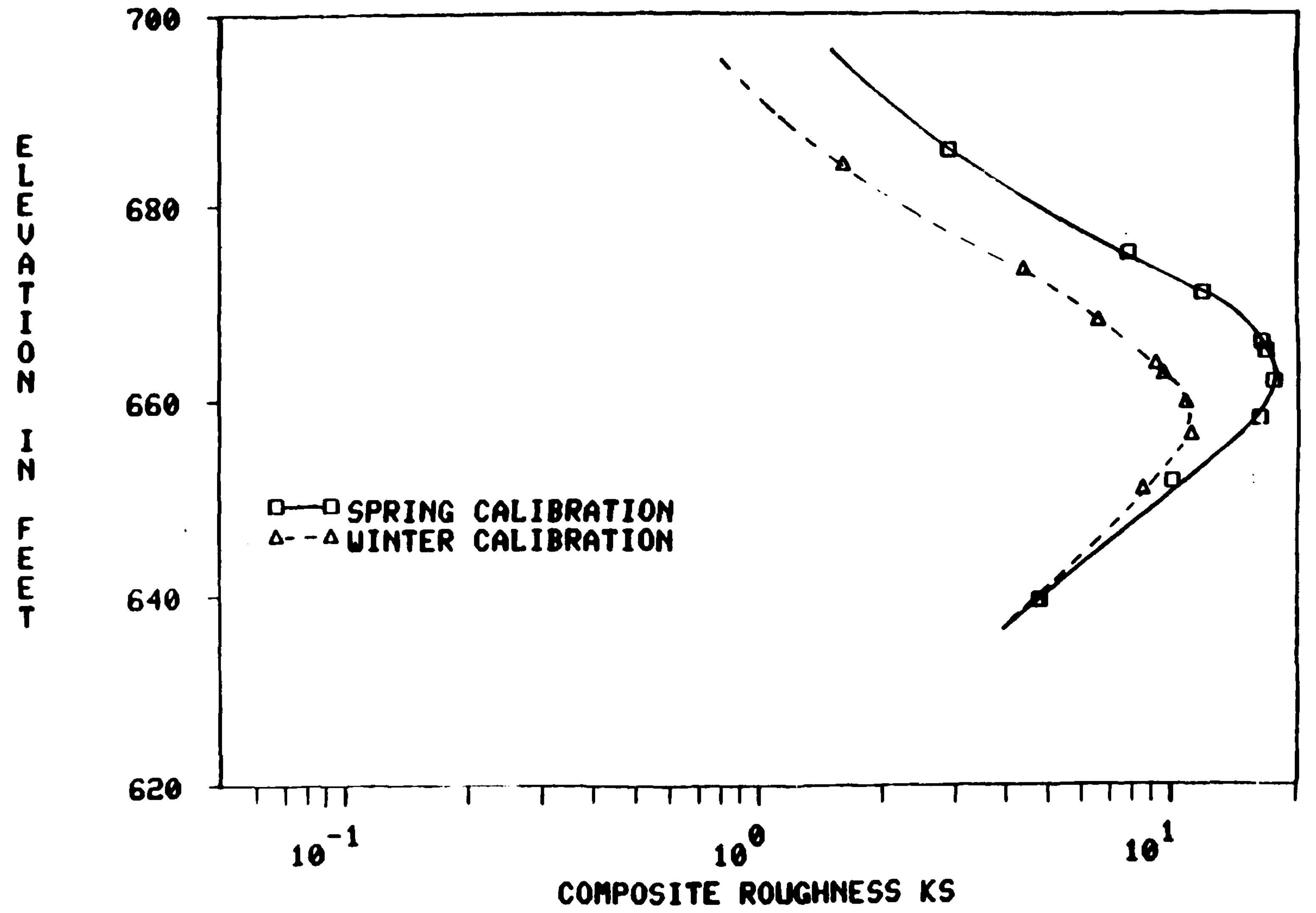

Figure 11. Roughness k_s versus water-surface elevation at Williamson gage, 94,000-cfs calibration

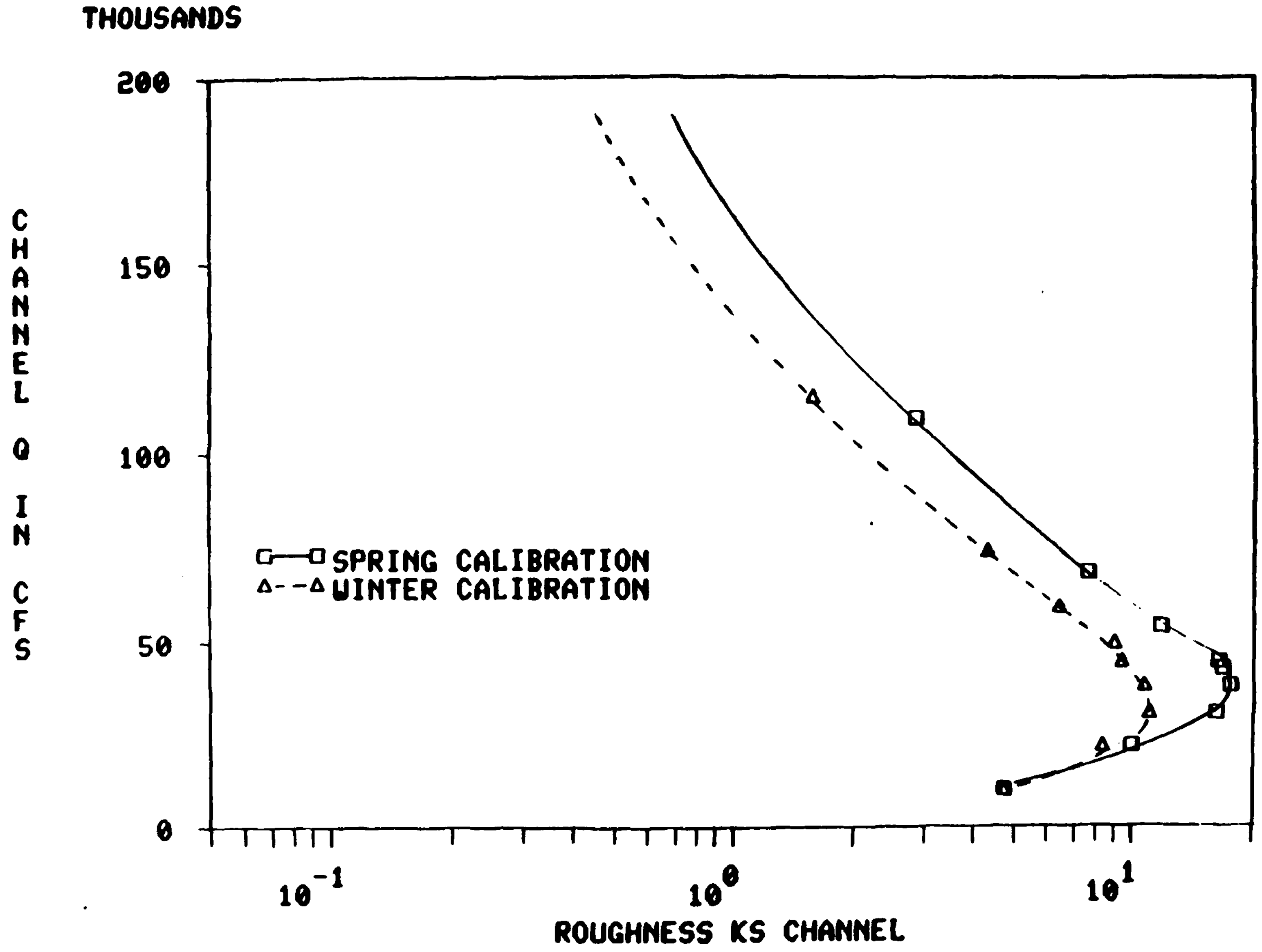

Figure 12. Roughness k_s versus channel discharge at Williamson gage, 94,000-cfs calibration

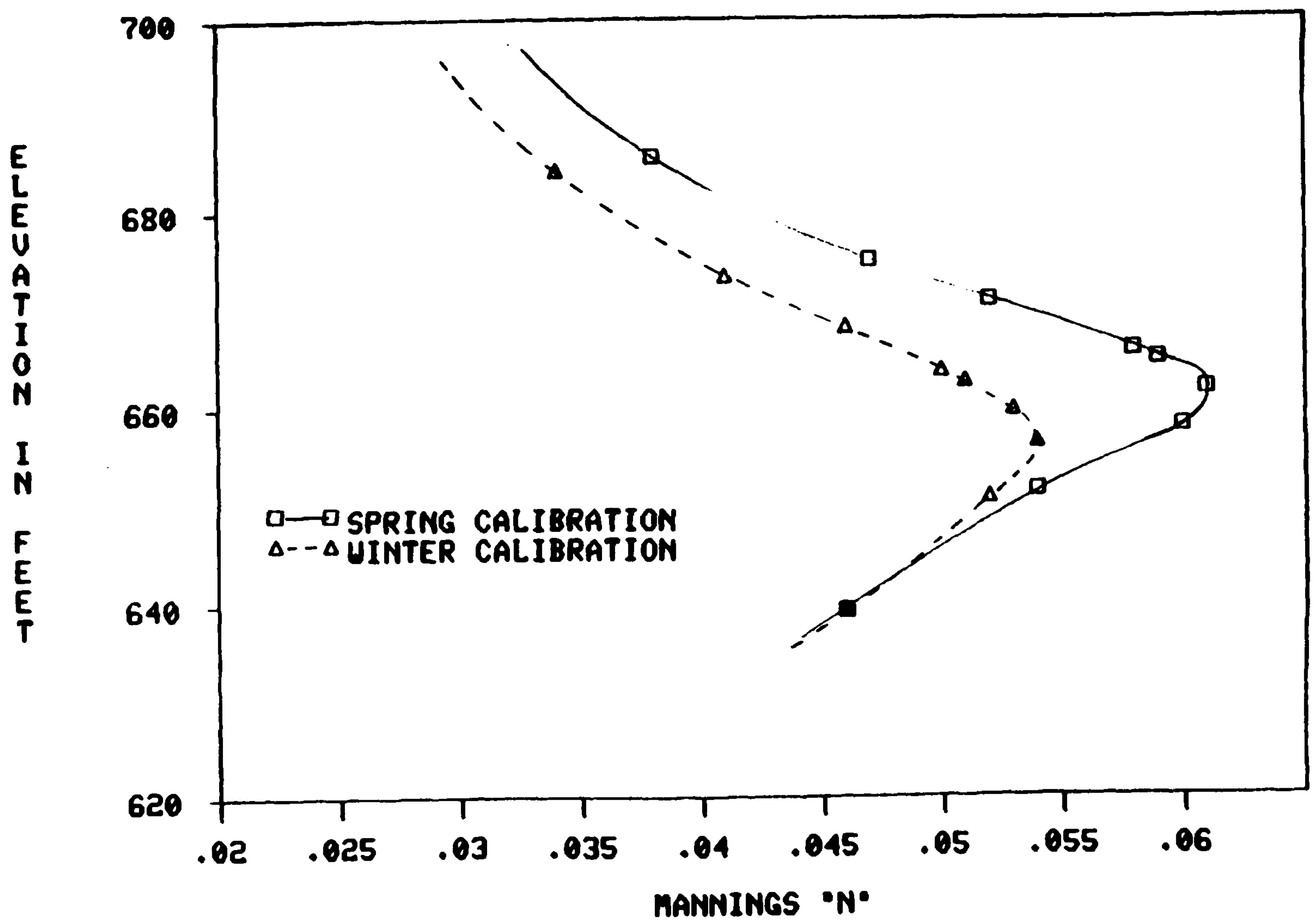

Figure 13. Manning's "n" versus water-surface elevation at Williamson gage, 94,000-cfs calibration

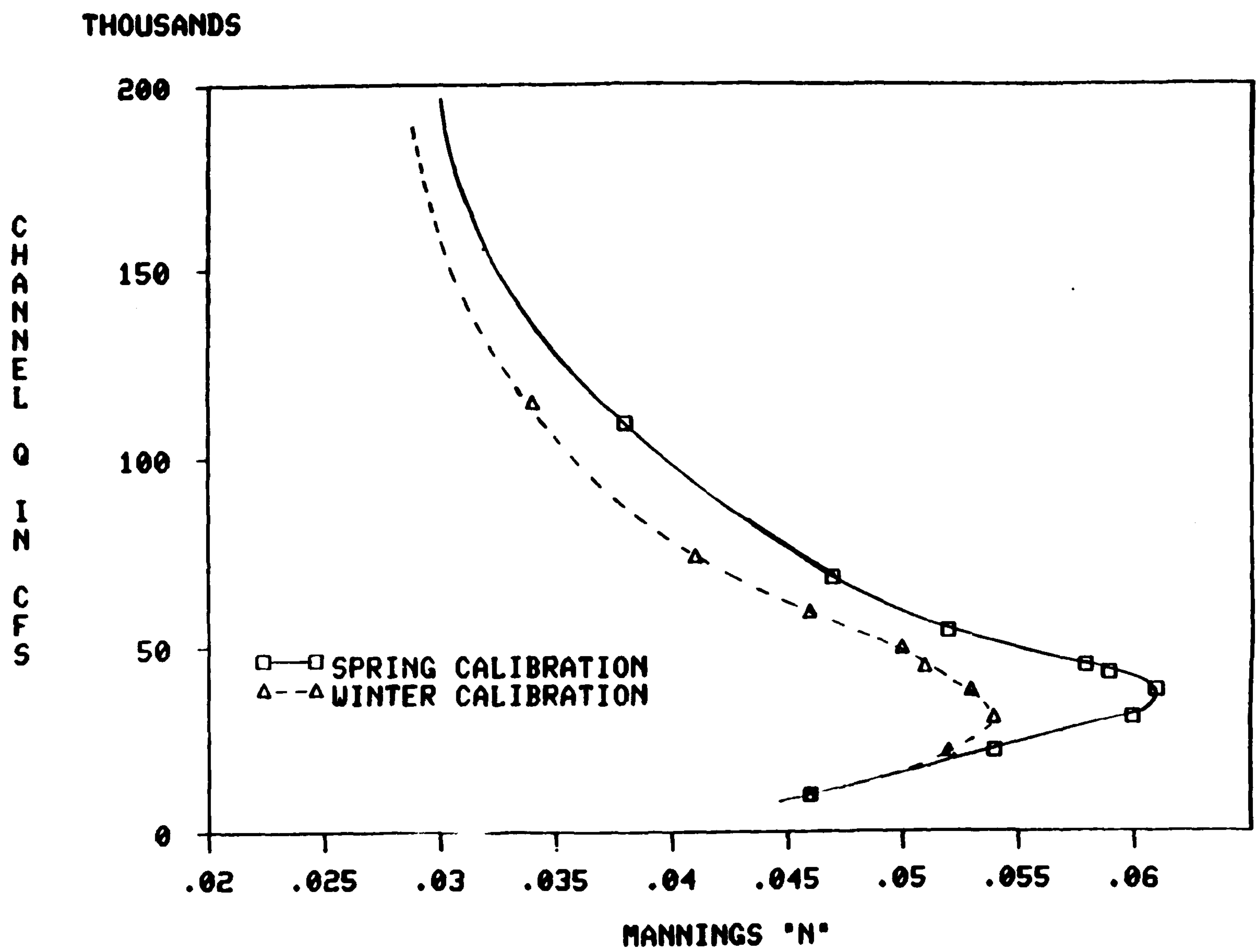

Figure 14. Manning's "n" versus channel discharge at Williamson gage, 94,000-cfs calibration

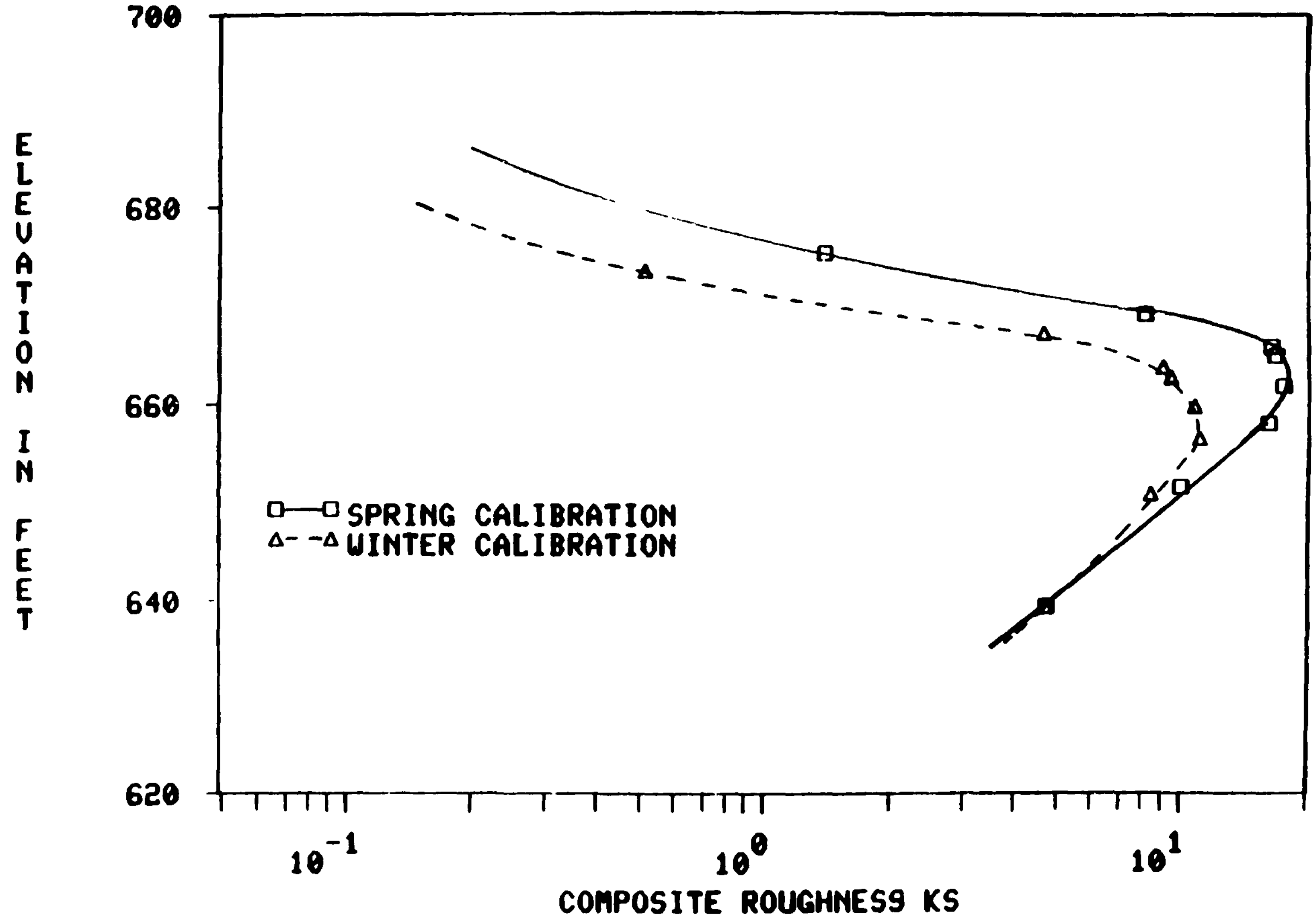

Figure 15. Roughness k_s versus water-surface elevation at Williamson gage, 117,000-cfs calibration

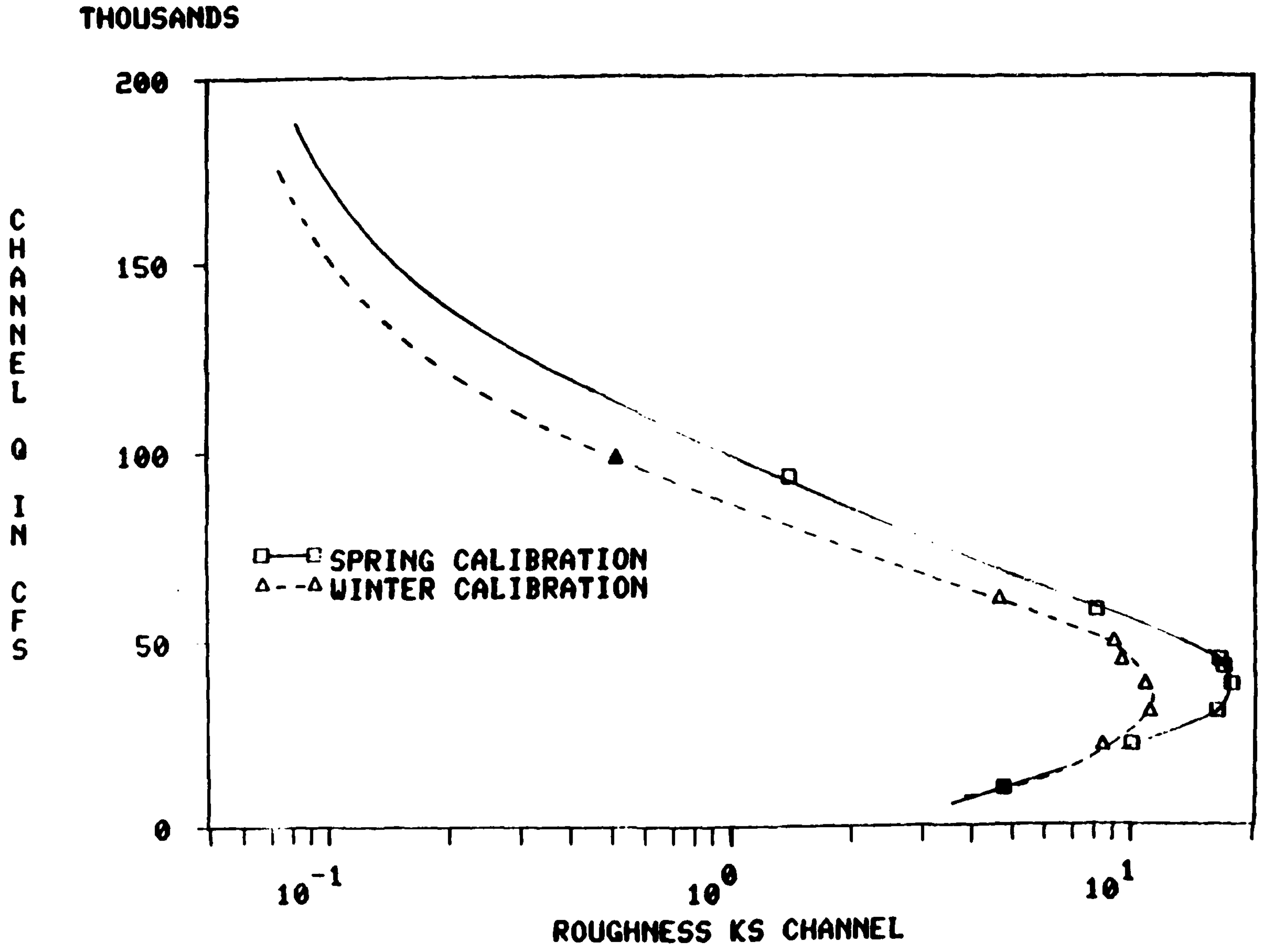

Figure 16. Roughness k_s versus channel discharge at Williamson gage, 117,000-cfs calibration

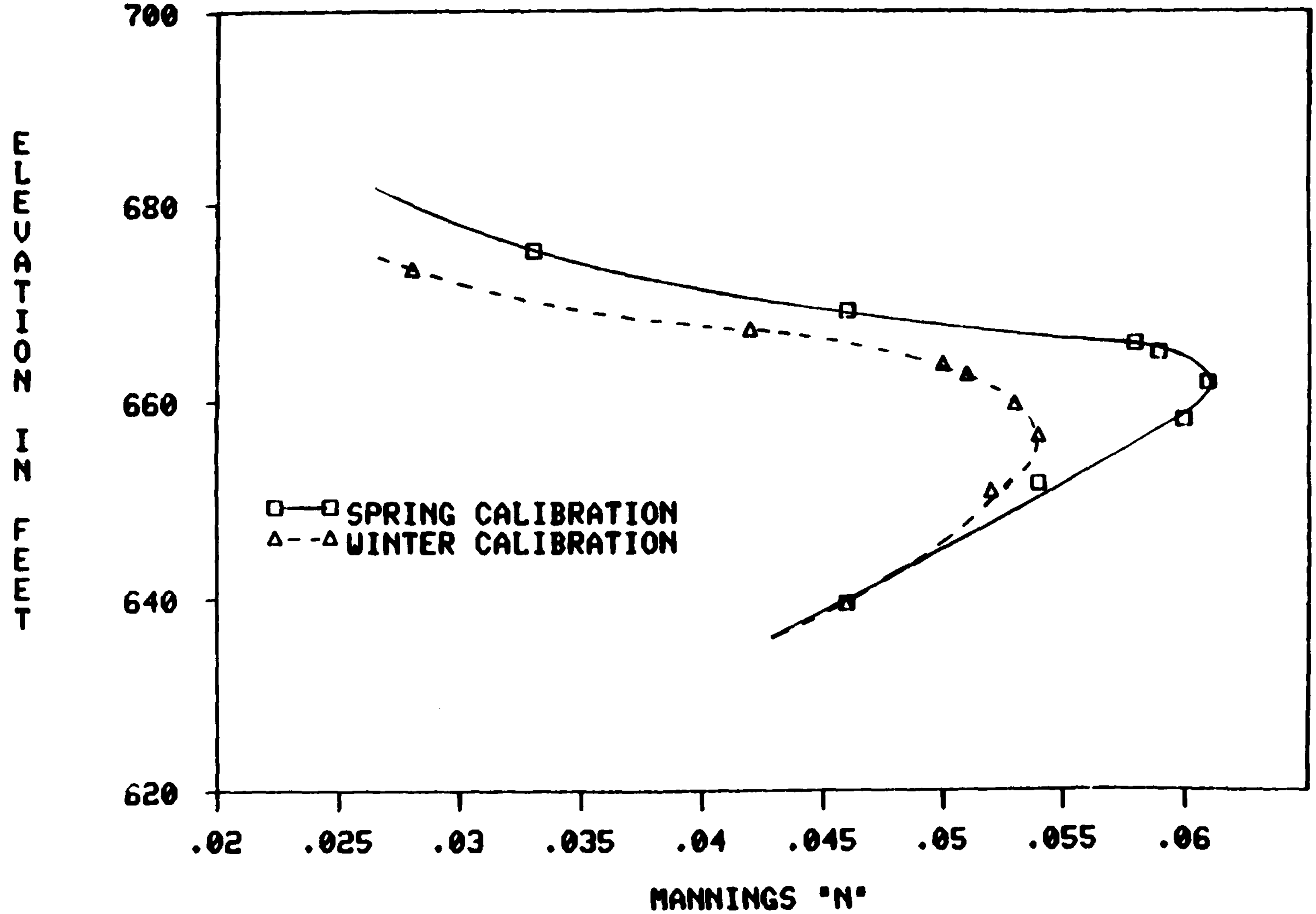

Figure 17. Manning's "n" versus water-surface elevation at Williamson gage, 117,000-cfs calibration

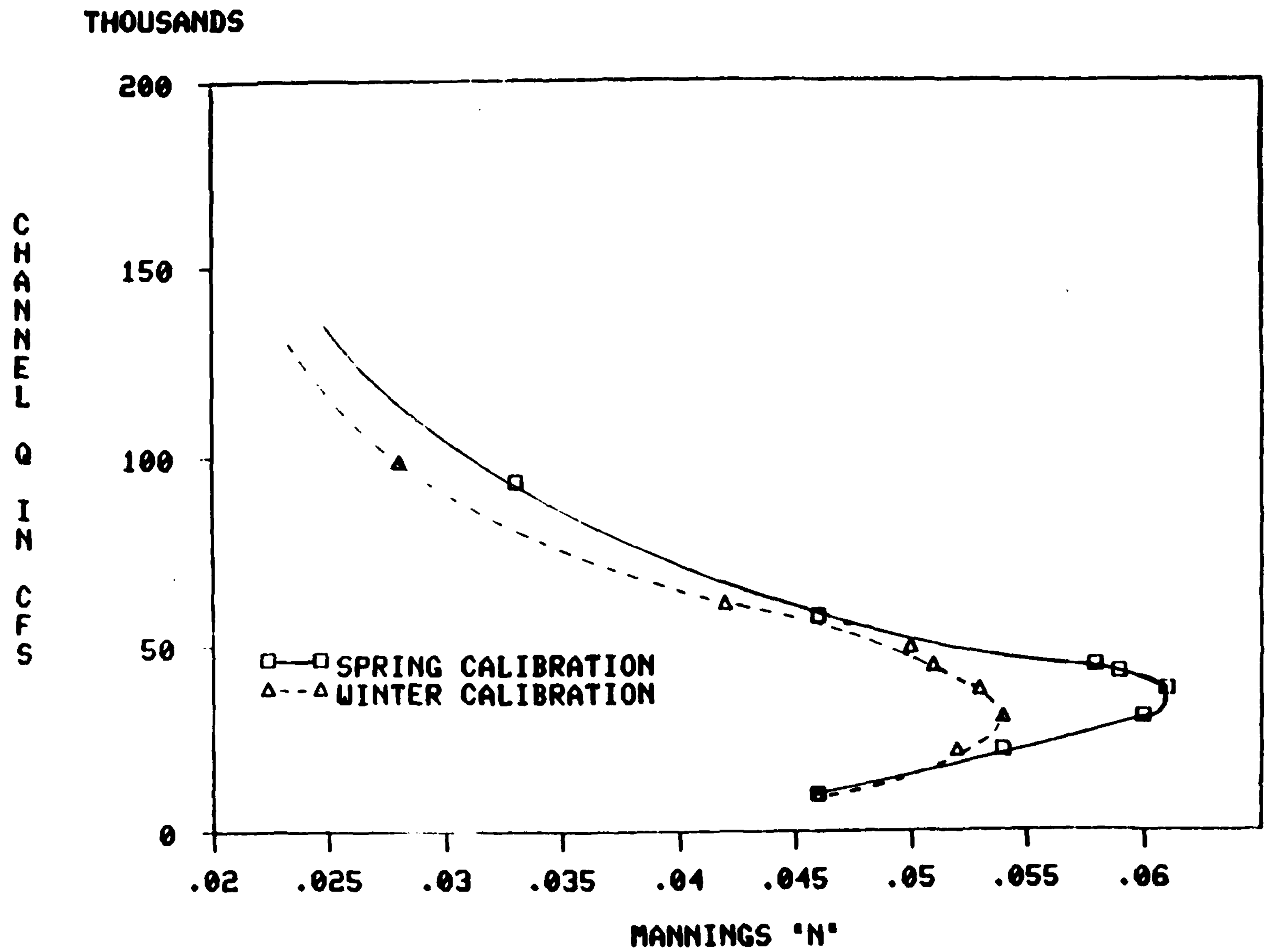

Figure 18. Manning's "n" versus channel discharge at Williamson gage, 117,000-cfs calibration

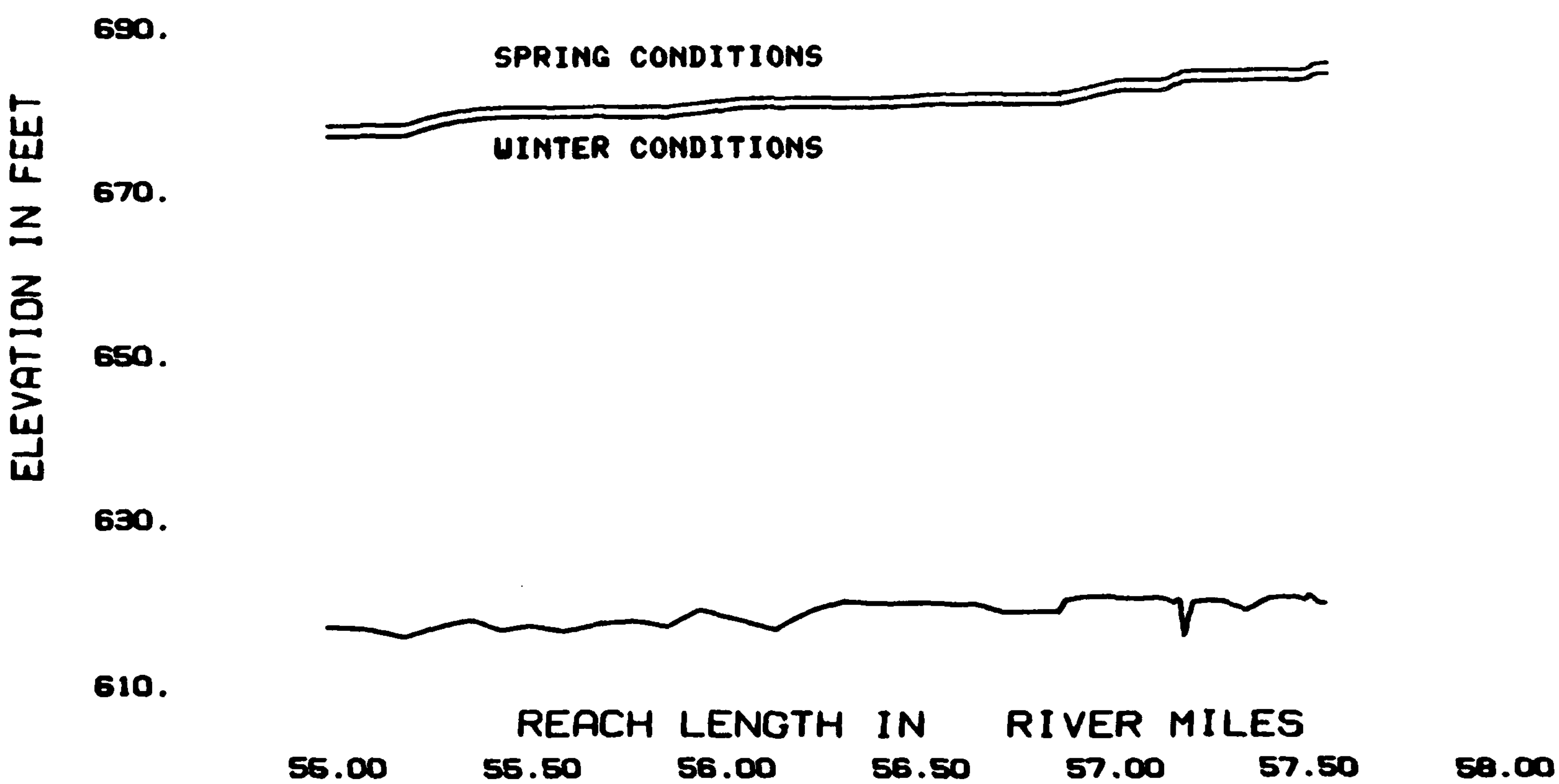

Figure 19. Existing conditions SPF water-surface profiles in Williamson area, 94,000-cfs calibration

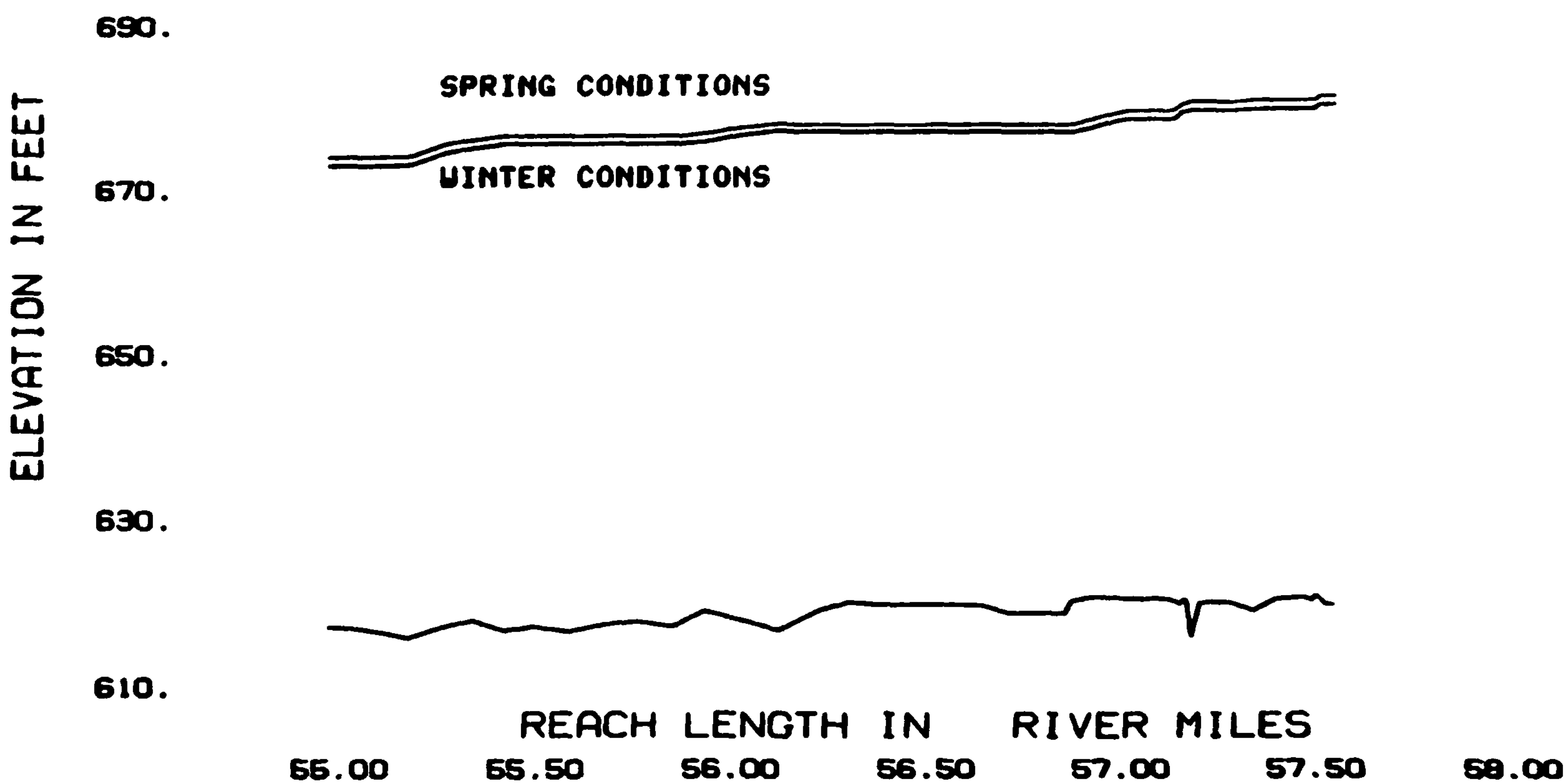

Figure 20. Existing conditions SPF water-surface profiles in Williamson area, 117,000-cfs calibration

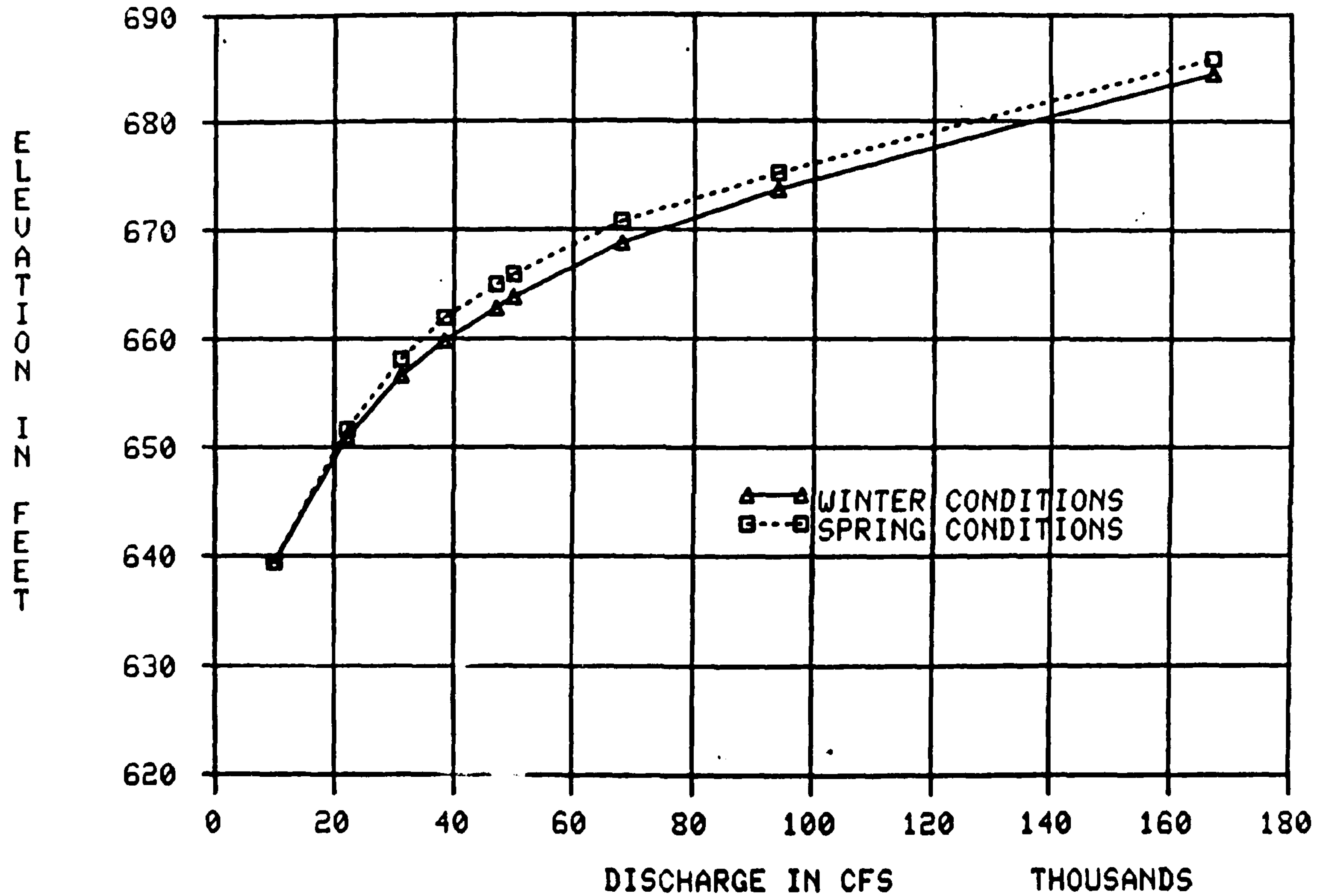

Figure 21. Project conditions rating curve at Williamson gage, 94,000-cfs calibration

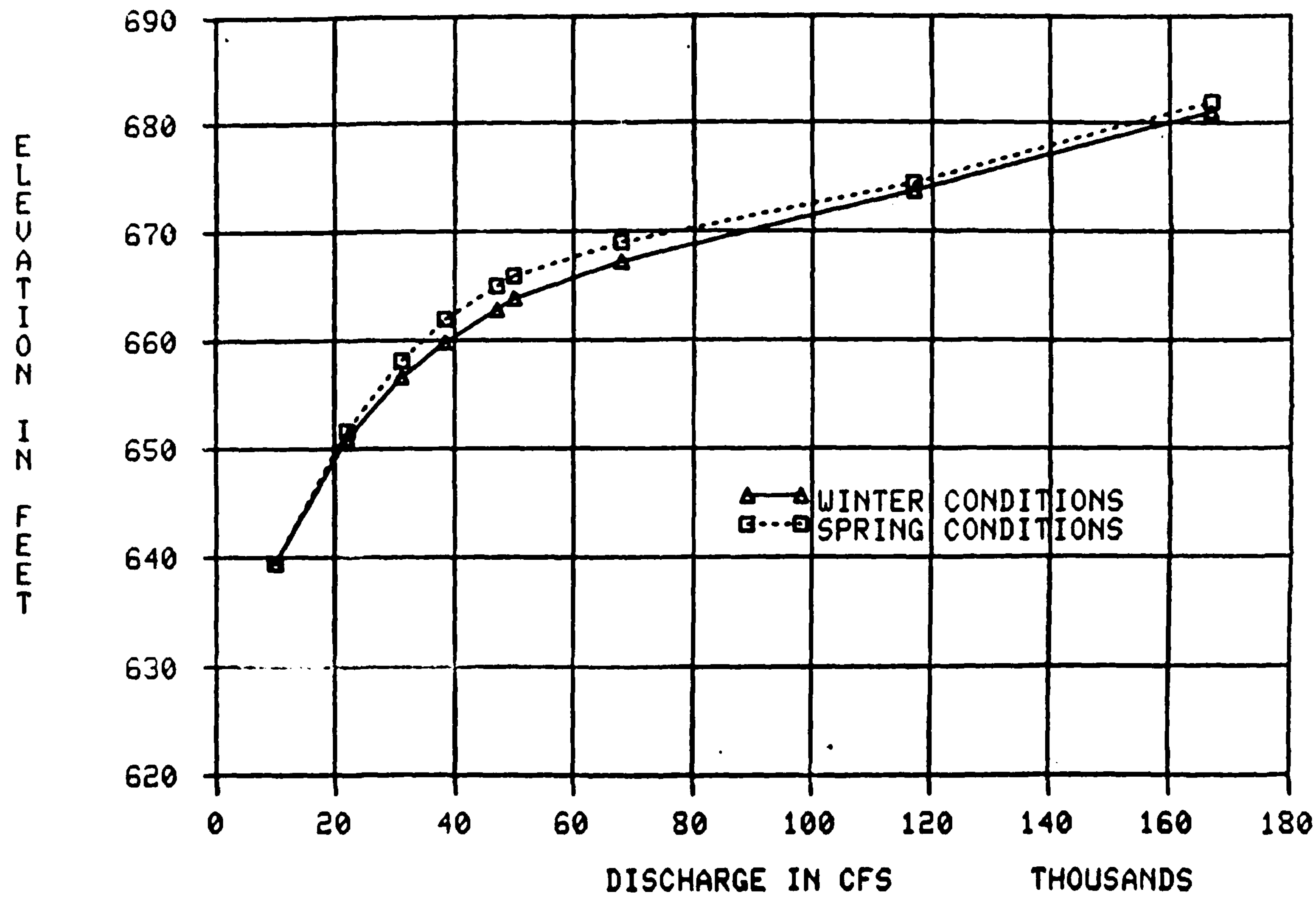

Figure 22. Project conditions rating curve at Williamson gage, 117,000-cfs calibration

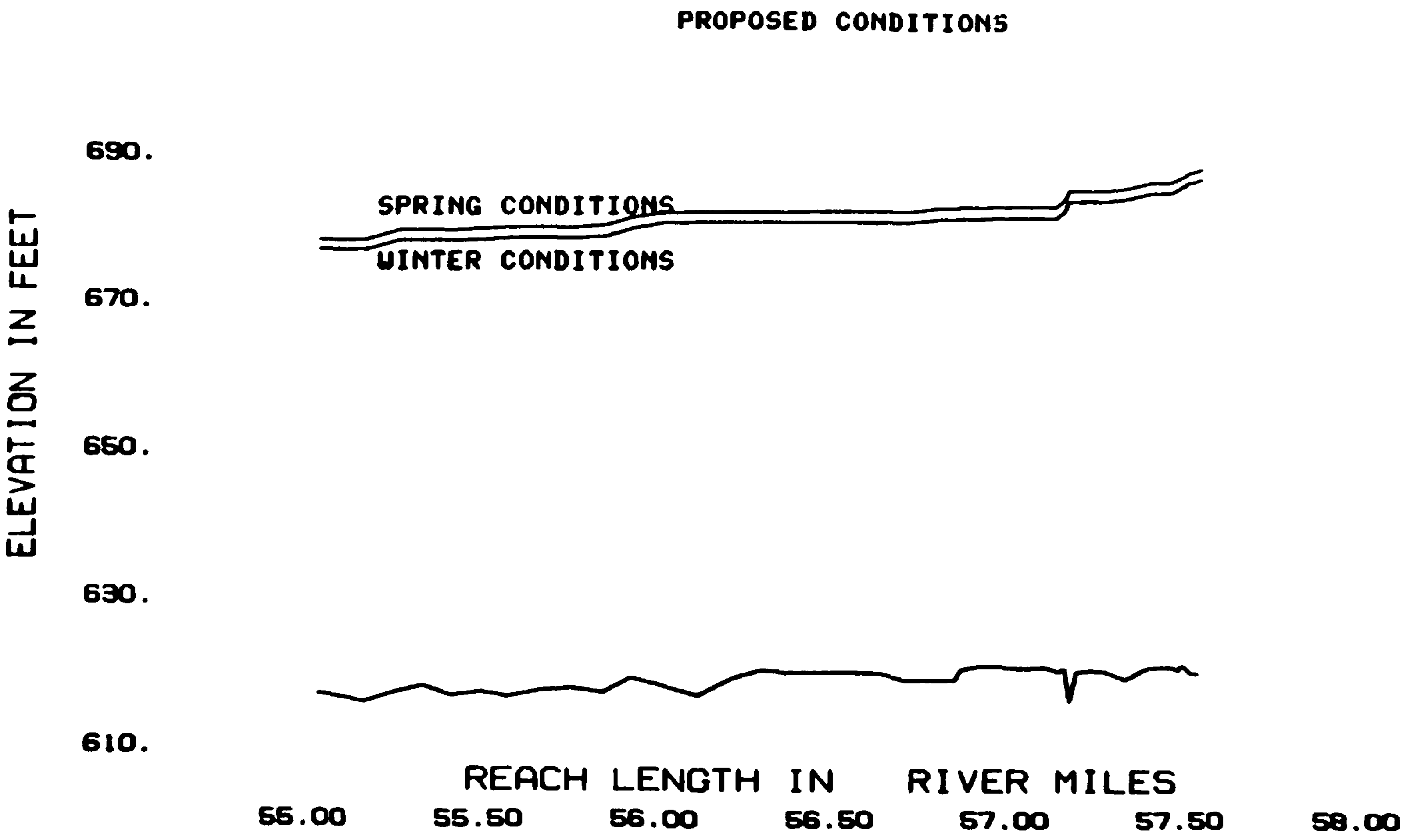

Figure 23. Project conditions SPF water-surface profiles in Williamson area, 94,000-cfs calibration

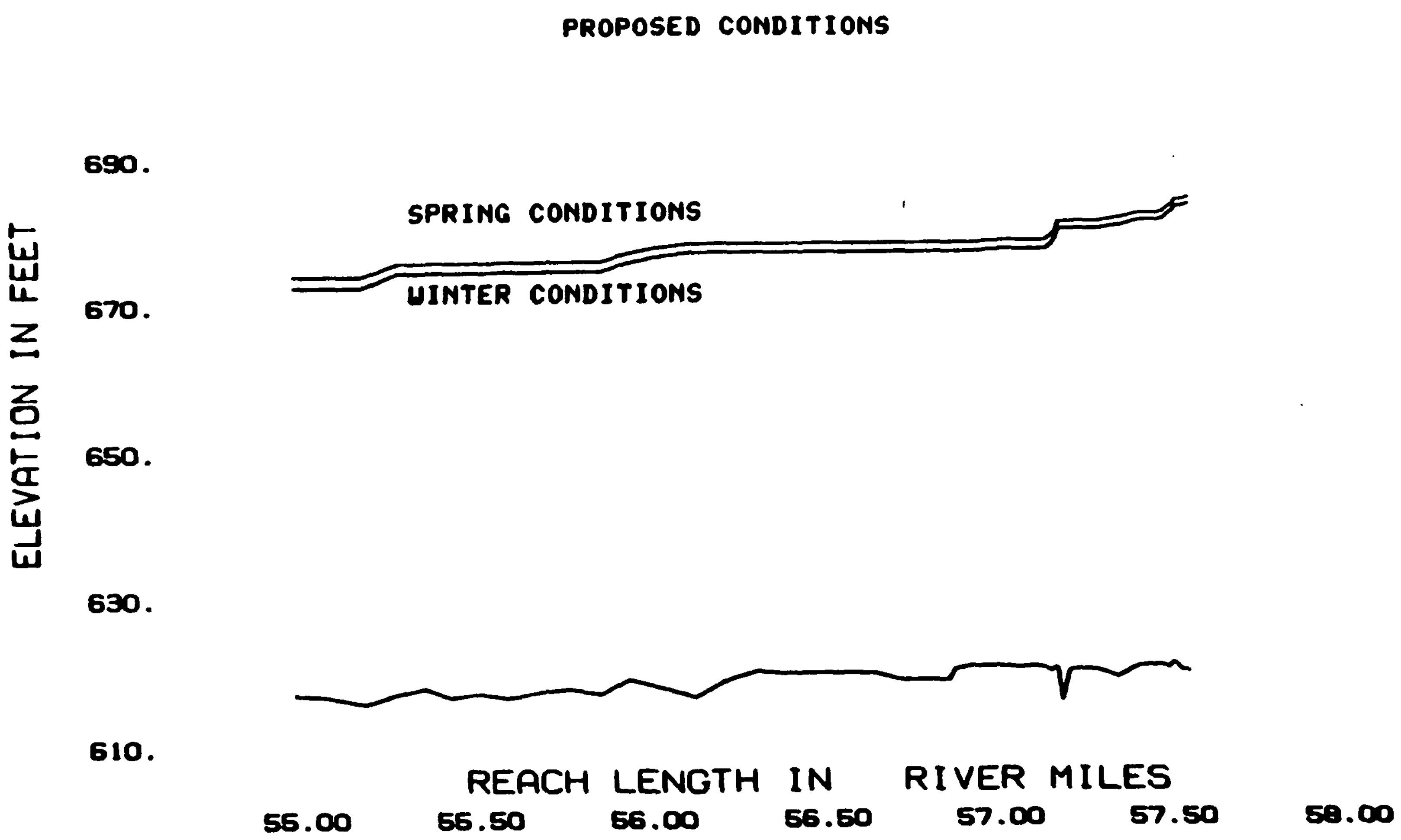

Figure 24. Project conditions SPF water-surface profiles in Williamson area, 117,000-cfs calibration

Appendix A

Computation of Composite k from Calibrated Manning's "n" From Hydraulic Design of Flood Control Channels, EM 1110-2-1601

$$C = 32.6 \log_{10}(12.2R/k), \qquad \text{hydraulically rough} \tag{1}$$

where

k = effective roughness height, ft

C = Chezy C

Also

$$\frac{C}{1.486} = \frac{R^{1/6}}{n} \tag{2}$$

Combining equations (1) and (2)

$$\frac{32.6 \log_{10}(12.2R/k)}{1.486} = \frac{R^{1/6}}{n} \qquad \text{or}$$

$$\log_{10}(12.2R/k) = (0.0456R^{1/6})/n \qquad \text{and}$$

$$12.2R/k = 10^{(0.0456R^{1/6}/n)} \qquad \text{and}$$

$$k = \frac{12.2R}{10^{(0.0456R^{1/6}/n)}}$$

From HEC-2 calibration of the 1984 flood, spring conditions

$n = 0.0584$

$R = 34.96$

$$k = \frac{12.2(34.96)}{10^{[0.0456(34.96)^{1/6}/0.0584]}} = \frac{426.5}{25.8} = 16.5 \text{ feet}$$

Appendix B

Determination of Minimum k

Tug Fork River at Williamson

Bed gradation samples were taken 27 March 1985 at Harvey Street Bridge, Williamson, W.V. (see Figure B1).

$$\text{Hydraulic radius} \approx 5 \text{ feet}$$

$$\text{Use } D_{84} \text{ of } 10 \text{ mm} = 0.033 \text{ ft}$$

$$\text{Use } D_{50} \text{ of } 2 \text{ mm} = 0.0066 \text{ ft}$$

Determine Manning's n

From Limerino (1970)

$$n = \frac{0.0926R^{1/6}}{1.16 + 2.0 \log(R/D_{84})} = \frac{0.0926(5)^{1/6}}{1.16 + 2.0 \log(5/.033)} = 0.0219$$

From Strickler (1923)

$$n = .034(D_{50})^{1/6} = 0.034(.0066)^{1/6} = .0147$$

Use n = 0.02

Determine k (assume hydraulically rough)

$$\text{From } \frac{1.486R^{1/6}}{n} = 32.6 \log (12.2R/K)$$

$$\frac{1.486(5)^{1/6}}{0.02} = 32.6 \log [12.2(5)/k]$$

$$\log [61/k] = 2.98$$

$$k = \frac{61}{10^{2.98}} = 0.064 \text{ , use } K = 0.07$$

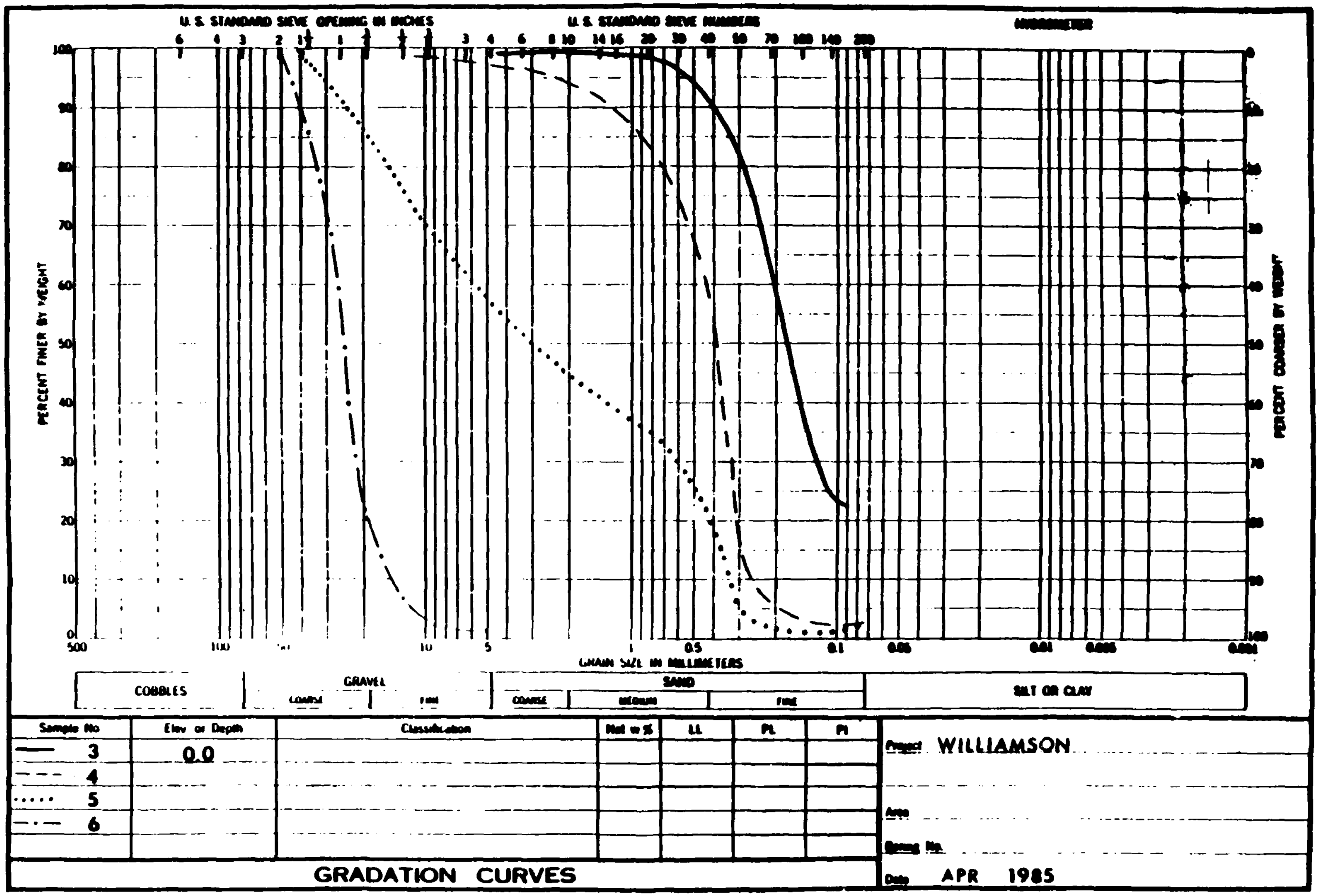

Figure B1. Bed gradations from Harvey Street Bridge, Williamson, W. Va., 27 March 1985

Appendix C

Computation of Composite k at Williamson, W. Va.

Cross-section 57.46 (ref. Hydraulic Design of Flood Control Channels, EM 1110-2-1601)

Discharge = 22,000 cfs

Water surface elevation = 651.7

a. Hydraulic elements

	Left Bank	Right Bank	Streambed	Summary
k	90.	75.	0.07	--
Area, A	502.	897.	4098.	5497.
Hydraulic Radius, R	11.2	11.8	31.5	--
*Chezy C	5.9	9.2	121.9	--
$CR^{1/2}$	19.8	31.7	684.2	735.7
$A(CR^{1/2}) \times 10^4$	0.993	2.84	280.39	284.23
$A(CR^{1/2})^3 \times 10^8$	0.0389	0.286	13100.00	13100.32
$A(CR^{3/2}) \times 10^8$	0.00111	0.00336	0.883	0.887

* $C = 32.6 \log(12.2R/k)$

b. $\bar{V} = Q/A$

$\bar{V} = 22000/5497 = 4.00$ fps

c. $S = .00034$ from HEC-2 calibration

d. $$\bar{R} = \frac{\Sigma A(CR^{3/2})}{\Sigma A(CR^{1/2})}$$

$R = 31.23$

e. $\tau_o = \gamma \bar{R} S$

$\tau_o = (62.4)(31.23)(0.00034) = 0.663$

f. $\alpha = \dfrac{A^2 \Sigma A(CR^{1/2})^3}{[\Sigma A(CR^{1/2})]^3}$

$= 1.73$

g. Effective k, α neglected

$C = \dfrac{\gamma \bar{V}^2}{\tau_o}^{1/2}$

$= 38.81$

from $C = 32.6 \log (12.2R/k)$

$k = \dfrac{12.2R}{10^{(c/32.6)}}$

$k = 24.52$

h. Effective k, α considered

$\alpha \dfrac{\bar{V}^2}{2g} = 0.43$

$V' = \left[(64.6)(0.43)\right]^{1/2} = 5.26$

$C = \left[\dfrac{(62.5)(5.26)^2}{0.663}\right]^{1/2} = 51.09$

$k = 10.32$

i. $n = \dfrac{R^{1/6}}{23.85 + 21.95 \log (R/k)}$

$n = 0.0516$

Q = 50,000

Water-surface elevation = 666.53

a. Hydraulic Elements

	Left Bank	Right Bank	Streambed	Sum
k	90.	75.	0.07	--
Area, A	1109.	1892.0	6023.0	9024.0
Hydraulic Radius, R	16.28	20.83	46.3	--
Chezy C	11.2	17.3	127.4	--
$CR^{1/2}$	45.2	78.9	866.6	990.7
$A(CR^{1/2}) \times 10^4$	5.01	14.92	521.97	541.91
$A(CR^{1/2})^3 \times 10^8$	1.03	9.28	39200.0	39210.0
$A(CR^{3/2}) \times 10^8$	0.00816	0.00311	2.420	2.450

b. $\bar{V}$ = 5.54

c. S = 0.000363 from HEC-2

d. R = 45.32

e. τ_o = 1.03

f. α = 2.01

g. Effective k , α neglected = 26.15

h. Effective k , α considered = 7.31

i. N = 0.046

$Q = 94{,}000$

Water-surface elevation = 675.68

a. Hydraulic Elements

	Left Bank	Right Bank	Streambed	Sum
k	90.	75.	0.07	--
Area, A	1617.0	2509.0	7212.0	11338.0
Hydraulic Radius, R	18.2	25.1	55.48	--
Chezy C	12.8	19.9	129.9	--
$CR^{1/2}$	54.5	99.8	967.7	1122.1
$A(CR^{1/2}) \times 10^4$	8.82	25.03	697.93	731.79
$A(CR^{1/2})^3 \times 10^8$	2.62	24.9	65360.0	65390.0
$A(CR^{3/2}) \times 10^8$	0.0161	0.0628	3.87	3.95

b. $\bar{V} = 8.29$

c. $S = 0.000352$ from HEC-2

d. $R = 53.99$

e. $\tau_o = 1.19$

f. $\alpha = 2.15$

g. Effective k, α neglected = 9.42

h. Effective k, α considered = 1.30

i. $n = 0.033$

Q = 166,950

Water-surface elevation = 685.8

a. Hydraulic Elements

	Left Bank	Right Bank	Streambed	Sum
k	90.	75.	0.07	--
Area, A	2686.0	3193.0	8527.0	14406.0
Hydraulic Radius, R	18.78	29.00	65.59	--
Chezy C	13.2	22.0	132.3	--
$CR^{1/2}$	57.3	118.3	1071.4	1247.0
$A(CR^{1/2}) \times 10^4$	15.4	37.8	914.0	[illegible]67.0
$A(CR^{1/2})^3 \times 10^8$	5.06	52.8	105000.0	105058.0
$A(CR^{3/2}) \times 10^8$	0.0289	0.109	5.99	6.13

b. $\bar{V} = 11.59$

c. S = 0.000322 from HEC-2

d. $\bar{R} = 63.42$

e. $\tau_o = 1.27$

f. $\alpha = 2.41$

g. Effective k , α = 2.52

h. Effective k , α considered = 0.11

i. n = 0.024

Summary

Q , cfs	22,000.0	50,000.0	94,000.0	166,950.0
A , ft^2	5,497.0	9,024.0	11,338.0	14,406.0
R , ft	31.23	45.32	53.99	63.42
$\bar{V}$, ft/sec	4.00	5.54	8.29	11.59
S , ft/ft	0.00034	0.00036	0.00035	0.00032
τ_o , lb/f^2	0.663	1.03	1.19	1.27
α	1.73	2.01	2.15	2.41
k , α neglected	24.52	26.15	9.42	2.52
k , α considered	10.32	7.31	1.30	0.11
Manning's n	0.052	0.046	0.033	0.024